Development of Optimization Models for Regional Wastewater and Storm Water Systems with Application in the Jizan Region, Saudi Arabia

by

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ABSTRACT

Imagine you live in a place without any storm water or wastewater systems!

Wastewater and storm water systems are two of the most crucial systems for urban infrastructure. Water resources have become more limited and expensive in arid and semi-arid regions. According to the fourth World Water Development Report, over 80% of global wastewater is released into the environment without adequate treatment. Wastewater collection and treatment systems in the Kingdom of Saudi Arabia (KSA) covers about 49% of urban areas; about 25% of treated wastewater is used for landscape and crop irrigation (Ministry of Environment Water and Agriculture [MEWA], 2017). According to Guizani (2016), during each event of flooding, there are fatalities. In 2009, the most deadly flood occurred in Jeddah, KSA within more than 160 lives lost. As a consequence, KSA has set a goal to provide 100% sewage collection and treatment services to every city with a population above 5000 by 2025, where all treated wastewater will be used.

This research explores several optimization models of planning and designing collection systems, such as regional wastewater and stormwater systems, in order to understand and overcome major performance-related disadvantages and high capital costs. The first model (M-1) was developed for planning regional wastewater system, considering minimum costs of location, type, and size sewer network and wastewater treatment plants (WWTPs). The second model (M-2) was developed for designing a regional wastewater system, considering minimum hydraulic design costs, such as pump stations, commercial diameters, excavation costs, and WWTPs. Both models were applied to the Jizan region, KSA.

The third model (M-3) was developed to solve layout and pipe design for

storm water systems simultaneously. This model was applied to four different case scenarios, using two approaches for commercial diameters. The fourth model (M-4) was developed to solve the optimum pipe design of a storm sewer system for given layouts. However, M-4 was applied to a storm sewer network published in the literature.

M-1, M-2, and M-3 were developed in the general algebraic modeling system (GAMS) program, which was formulated as a mixed integer nonlinear programming (MINLP) solver, while M-4 was formulated as a nonlinear programming (NLP) procedure.

This dissertation is dedicated to

my mother and father

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1 INTRODUCTION

1.1 Background

Water is a primary natural resource and humans and animals cannot live without it. In many parts of the world, water is under threat, causing many social, political, and economic problems. Recently, the importance of water has been widely recognized, and the need to provide good quality, non-polluted water is a focus of this research area. Adopting an integrated and/or sustainable water resource planning and management may protect water users from the impact of many factors on freshwater resources now and in the future. According to global experiences, researchers have agreed that the aggregation of many factors that work together lead to the deterioration of water quality and quantity. Climate change, for instance, already works as a direct factor on the global hydrological cycle, having a negative reflection on the availability and continuity of renewable water supplies on this planet for various water uses.

Researchers have also agreed that the drivers affecting and having a negative reflection on both water quality and quantity on this planet are population growth, industry, economic, technology, and social and legislative conditions. These factors are leading to an increasable competition among users, which might generate conflicts. This provides an excellent motive for many countries to work together or separately to maintain both the quantity and quality of the available water resources. High economic nations that already have integrated water management systems are working to deflect the challenge, regardless of the cost of sustainability in their solutions and measures. In some other countries, the economic, social, and environmental factors have significant impacts and costs are essential to maintaining their water requirements considering the mentioned standards.

Water reuse systems have been practiced in many ways since early urbanization. But designed wastewater systems have only recently been studied and have received little practice in the wastewater community. Today, efforts in the research and development of wastewater systems have given it an important role in those areas where the availability of water sources cannot satisfy the demand, such as in arid and semi-arid regions. Urban population growth is continuing at an unprecedented rate, causing severe problems in water planning. Wastewater systems affect the quantity and quality of the reuse water system directly.

The Kingdom of Saudi Arabia (KSA) has been growing fast in industry, population, and urban development; therefore, the demand for water is increasing in both quality and quantity. General organization for statistics of KSA estimated that the population of KSA in 2016 was approximately 31.74 million, of which Saudis number 20.08 million people (63.3%) and non-Saudis number 11.66 million people (36.7%). Approximately 11.74 million Saudis under the age of 30 represent 58.5% of the total Saudi population, and the number of those under the age of 44 is 80.8%, with an estimated total population of 16.2 million. Thus, Saudis are concentrated in the age group of less than 44 years, which indicates that the youth are essential to this stage of the history of KSA. In 2017, the quantity of drinking water reached 3150 million cubic meters (MCM) in all regions in KSA. The annual growth rate of drinking water distributed by the primary sources is increasing every year. In 2007, the annual consumed drinking water was about 1977 MCM, and in 2017, it reached 3150 MCM, a 60% increase (Ministry of Environment Water and Agriculture [MEWA], 2017).

Pollution is relevant and can affect water in populated areas or areas with industrial activities. Wastewater systems require wastewater sources, collection, and wastewater treatment plants (WWTPs). To achieve a sustainable solution system and developing a new idea that has not been done before, comprehensive work should be investigated. Sustainability of the system can be described by providing a different solution to approaches that can allow decision-makers/stakeholders to obtain the purpose of it and give an idea of changing the behavior of society would have a significant effect on the solution.

1.2 Problem Statement

For any wastewater and storm water system, there are two essential components: the sewer network and treatment plants. It also can be described as a given number of wastewater discharges, the type and concentration of pollutants at sources, and the candidate locations and sizes of potential collection and WWTPs. Further, to convey wastewater from sources to WWTPs, locations and designs of sewer networks must be found, so the total cost of installing, operating, and maintaining infrastructures can be minimized. In other words, the optimization of regional wastewater and storm water system models should be formulated to identify the location and design of sewer systems and WWTPs in a large-scale region that satisfy the hydraulic constraints at a minimum cost.

The problem statement can be summarized in four categories. First, the development of new sustainable optimization models for the problem must be formulated and tested by one or more optimization programming codes. Second, the importance of using the proposed models for arid and semi-arid regions must be discussed. Third, reasons for using the iso-nodal line (INL) concept and how that helps solve the problem must be given. Fourth, this study investigates the proposed models by applying them to a real-world problem.

1.2.1 Optimization Models for Storm Water Systems and Regional Wastewater Systems

Wastewater is considered to be a treatable water resource that could be used in different applications. It is regarded as a primary source of water pollution in urban areas (Cunha et al., 2009). According to the fourth World Water Development Report, over 80% of global wastewater is released into the environment without adequate treatment (UNESCO, 2017; United Nations Water [UN Water], 2017). For instance, around 80 % of domestic wastewater in KSA is discharged into septic tanks (Amin et al., 2015).

As a consequence, the demand for water is increasing strong in a country with limited natural water resources and an arid climate (Abderrahman, 2001). However, optimization techniques can be used to solve many types of water collection problems for operation and design, as well such as for collection or branched type, looped and pressurized, or gravity flow system (Mala-Jetmarova et al., 2017). For regional wastewater system planning, the optimization approach has been used since the early 1960s. According to Brand and Ostfeld (2011), there are a few optimization models that include the entire system, such as wastewater sources, sewer pipe network, treatment plants, and end disposals/users, in one optimization model.

The aim of planning regional wastewater systems is to find the minimum cost for the layout and pipe design that can dispose of the wastewater from urban areas. Consequently, the hydraulics and the water quality standards of the treated wastewater should be satisfactory after leaving WWTPs. In other words, regional wastewater systems work to assure that wastewater is collected and treated to meet the minimum hydraulic constraints for minimum cost. The optimum solution is defined by a possible layout and design of the sewer system that meets the environmental and technical standards the most economically. However, finding the optimal sewer diameter and treatment plant capacity for an individual system is an exhausting process. This is due to the fact that wastewater systems are costly and difficult to inverse, that water quality standards must be met, and that they need complex planning processes. Some models are dealt with at the local level for each city or part of a city; regarding costs, sustainable economic criteria and environmental performance at a regional planning approach can provide a better solution. To move toward a more sustainable strategy for urban wastewater management, the operation and design of sewers and treatment works should be integrated (Vollertsen et al., 2002).

The problem under consideration is how to select the layout and design of a sewer network and WWTP at minimum cost. WTTPs collect and drain wastewater from different cities with a given number of collection/sources/WWTP nodes located at various points in the region. The design inflows at each source node are known from statistical analysis, such as population size. Optimization models have the ability to decide where the connections of the nodes should be and the pipe design (crown elevations and diameter) for each connection. The optimization model for layout and pipe designs should be developed simultaneously, because they are dependent upon each other. Mays (1976) identified the main tasks in the development and construction of such an optimization model:

1. Representing a set of sources, collections, and WWTP/outlet nodes for unpredictable topographic conditions in a form suitable for digital manipulation.
2. Selecting the solution method and optimization program that can handle design constraints, various forms of cost functions, and hydraulics of flow.

These primary objectives must be regarded as copulative in order to arrive at a solution plan that can be effectively built and used to design large sewer networks. A brief detail of the sewer network design is presented in Figure 1.1. Mays (1976) explained that most storm water systems are dendritic or tree type systems, which are defined as follows:

1. external nodes (sources/manholes) at which only one branch begins.
2. internal nodes (collections/sources/manholes) or junctions of sewers where two or three links meet.
3. root node(s), i.e., outlet(s) of the sewer system, which can be represented as candidate WWTP or outlet nodes.
4. branches that connect nodes without the formation of closed routes or network loops, i.e., system sewers.

A typical tree-type sewer systems node-link (manhole/source/outlet-sewer) depiction is essential in formulating the optimization model.

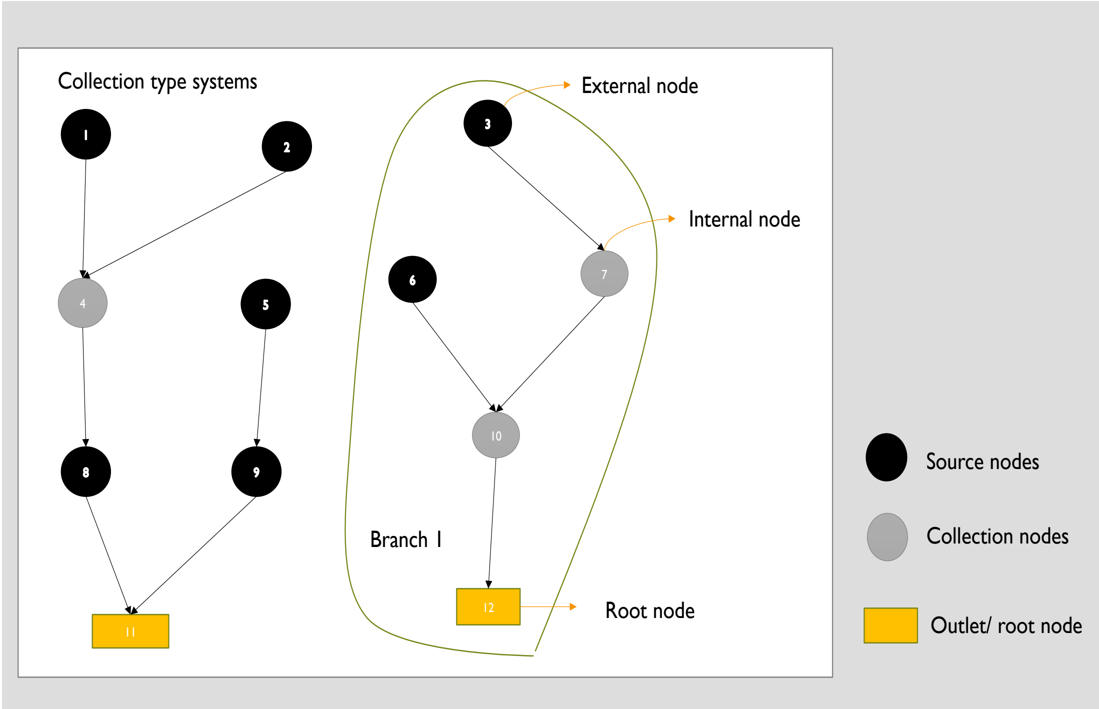


Figure 1.1. The Collection Type System (adopted from Mays, 1976).

1.2.2 Population Growth

As mentioned above, the sustainability of any wastewater/storm water collection system faces the problem of meeting the future demand using existing water supplies. World demographics have faced several changes due to human population dynamics for a long time. Populations can change through three processes: the percentage of births in the country, the death rate, and migration. The percentage of births and death rate are responsible for the continuing population growth in the world today, originating in developing countries. Migration results from the regional or international resettlement of a population or the movement of people between rural and urban areas. Because of urbanization, the urban population has risen from about 10% of the world population at the beginning of the 20th century to more than 50% today (United Nations [UN], 2018). Suburbanization is also gaining relevance in some developed countries. The migration processes associated with population growth has resulted in particularly intense population dynamics. The population is increasing in states such as Arizona, California, and Florida, while rural area populations are decreasing in developed countries due to migration rates and reduced birth rates (Spencer & Altman, 2010). For example, the population in Japan is expected to decline approximately 20% by 2050 (Kopf, 2018).

1.2.3 The Iso-nodal Line (INL) Method

The INL method can be used to solve any system that has flows from upstream (I) nodes to downstream (I + 1) nodes, with no flow between two nodes in the same INL. In general, this concept can be used for any system that has a dendritic or tree-type network (e.g., transportation, graphs, and social networks). An INL is defined as an imaginary line connecting nodes that have the same number of pipes connecting to the outlet of the drainage system. It can handle any layout system problem, as shown in Figure 1.2. There are various types of collection systems, from the local level, as in storm sewer systems, to the regional level, as in wastewater systems.

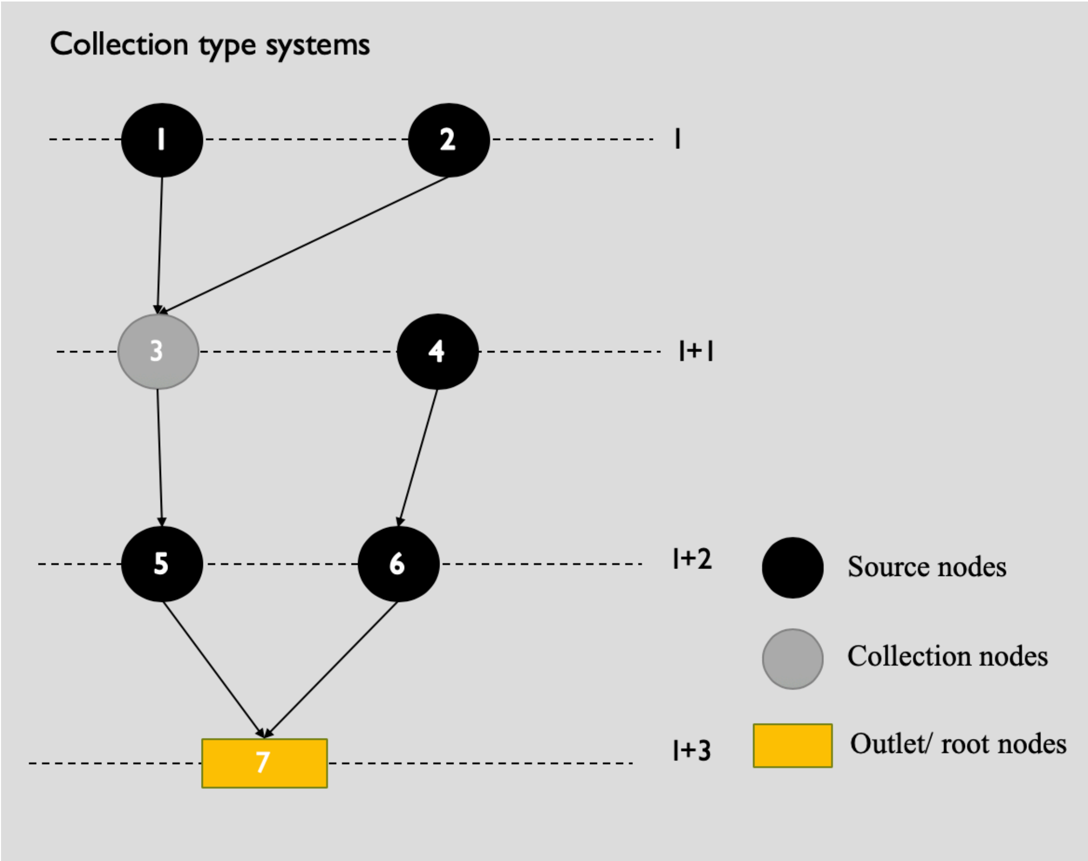


Figure 1.2. The Iso-nodal Line Method.

Section 2.4 in Chapter 2 provides detailed information about previous models based upon the INL method for solving water resource problems.

1.3 Research Objectives, Goals, and Phases

The minimum design cost of collection systems (storm water and regional wastewater systems) is determined through the following:

1. Optimization of the system layout.
2. Optimization of the hydraulic pipe and wastewater treatment plant design.

These two models form regional wastewater and storm sewer system design. The models are nonlinear and discrete in nature, with hydraulic and topological constraints. Due to the complexity of optimizing the layout and pipe design simultaneously, very little research is found in the literature regarding regional wastewater system design optimization. Reviewing previous related models indicated that the issue of determining the combined optimal layout and design of a collection system (e.g., a storm sewer system) using the INL method has been solved using discrete differential dynamic programming (DDDP). A detailed literature review for regional wastewater system design optimization and the INL method is given in Chapter 2. Many of the previous developed optimization models that minimize the cost of the layout and hydraulic design of the system were restricted by at least one of the following issues: (a) parameters included in the cost functions, (b) the difficulty in considering commercial pipe diameters, (c) inadequate hydraulic considerations of the of flow, and/or (d) inefficient application to large-scale design problems.

The proposed study is an attempt to the addressee subjects that have not yet been covered in the literature of regional wastewater and storm sewer system design. Although the significance of the planning and/or design problem of regional wastewater systems has been identified and addressed for more than half a century, little work has been done using the INL method. The proposed research presented here extends the problem of finding an optimal solution for the infrastructure configuration in regional wastewater and storm sewer systems. This was implemented using a modern approach, involving more realistic and state-of-the-art optimization models and solution methods. This dissertation falls into two primary approaches: 1) location of system components and 2) location and design for system components. These are described in the chapters that follow.

1.3.1 Research Goals

The research goals addressed in this dissertation are to develop and improve sustainable optimization models for regional wastewater systems and storm water systems while minimizing costs. The models applied to the Jizan region, KSA are based on adopting optimization models using the INL method. The models considered storm water systems to be another part of the research.

In this dissertation, established optimization models deal with planning a regional wastewater system and the hydraulic design, such as pump stations, commercial diameters, etc. An early step in the developed optimization model provides a practical solution method that helps solve upcoming models and provides an indication of the proposed approaches. Furthermore, another optimization model was developed to simultaneously determine the layout and pipe design for storm water systems. Therefore, the development of a design model for given layouts of storm sewer systems has been achieved. As a result, the hypothetical application for each developed model has been implemented to prove the accuracy of and to test the computed results. Finally, the proposed models are applied to case studies of regional wastewater and storm sewer systems.

1.3.2 Research Phases

The work phases of the proposed research are categorized into two types: the development of the models and their application.

1.3.2.1 Phases of Model Development

Phase 1

The development of a mixed integer nonlinear programming (MINLP) optimization model for the layout and locations of wastewater system, considering technical, environmental, and economic parameters, should be studied. This model will address regional wastewater system problems in a way that reflects the needs of both decision-makers and consumers. More specifically, the primary goal of phase 1 is to develop a deterministic approach to regional wastewater systems, which searches the optimal configuration solutions for sewer networks and WWTPs.

In this phase, the proposed optimization models will be based on the performance and flexibility of the system, influenced by layout and locations of WWTPs of the system and considering the minimum cost of flows, total length, and treatment capacity. These proposed models have been developed and applied as simple examples to assess the technical sustainability of a wastewater system. Furthermore, economic parameters have been identified that affect the total costs (i.e., construction, operation, and maintenance) of wastewater system elements. This phase is referred to as M-1.

Phase 2

The development of phase 2 is an expansion of the above optimization model (M-1) to take into account the hydraulic design of regional wastewater systems. The idea has already been proposed by Mays and Tung (1992) for designing branch systems for agriculture purposes. Our design system would include location, size, type of pump station, if necessary, size and type of commercial diameters, and WWTP. This phase is referred to as M-2.

Phase 3

In phase 3, we develop an MINLP model to simultaneously determine the hydraulic pipe design problem and layout problem. The objective of phase 3 is to develop an optimization model for the simultaneous layout and pipe design of storm sewer systems. The model aims to provide minimum costs that satisfy the minimum requirement of commercial pipe diameters, pipe slopes, and excavation works for the system and to provide the decision-makers or stakeholders with another alternative solution for the layout and location of the system. This phase consists of an optimization procedure to consider many possible designs (combinations of crown elevations, slopes, and commercial diameters) for possible network configurations at each node of the system. In addition, the connectivity model considers possible network configurations at each manhole and selects the cheapest route based upon the network costs for the upstream manhole, the node under consideration, and the succeeding downstream node.

This difficulty can be resolved by good engineering judgment and simple testing of the model to determine various model input parameters and the proper selection of candidate locations of collection or outlet nodes. However, phase 3 has been completed and there are extensive offers and new ideas, as discussed in chapter 8. This phase is referred to as M-3.

Phase 4

In phase 4, we develop a nonlinear programming (NLP) model to solve the hydraulic pipe design problem for given layouts. The objective of phase 4 is to develop an optimization model for the pipe design of storm sewer systems. The model aims to provide minimum costs that satisfy the minimum requirement of commercial pipe diameters, excavation work, and velocity constraints for the system. This phase consists of an optimization procedure to consider many possible designs (combinations of crown elevations, slopes, and commercial diameters) for given network configurations at each manhole of the system. This phase is referred to as M-4.

1.3.2.2 Phases of Models' Application

This dissertation applies the above-proposed methodologies to a case study of real-world problems in an arid and semi-arid climate. Some parts of the world do not have surface water that can receive treated wastewater, which makes the problem more difficult to address, so the risk of wastewater should be minimized as well. Treated wastewater body is considered to be an alternative source for regions facing water shortages. The proposed methodologies can be applied to different parts of the world, such as Africa, Australia, Southeast Asia, and the Middle East. These methods (M-1 and M-2) are applied to the Jizan Region of KSA, which is an arid country and has been selected as a case study for this dissertation. Considering optimization techniques as a solution for planning water problems in KSA is one of newest topic areas that many researchers have not taken into consideration. This dissertation applies M-3 and M-4 to storm sewer systems that include optimal layout and pipe design in the regions and that have frequent storm events, causing flood damage. It is considered to be one of the most critical pieces of infrastructure for modern cities, which must be designed for minimum costs. According to Guizani (2016), in each event of flooding, there are fatalities. In 2009, the most deadly flood occurred in Jeddah, KSA, with more than 160 lives lost.

Phase 5

This dissertation applies each proposed model (M-1, M-2, M-3, and M-4) to hypothetical applications, with different scenario assumptions, and discusses the results to provide a better-quality solution method and performance. It decides which solution method should be considered for each model.

Phase 6

As shown in Table 1.1, the data collection and information for the Jizan Region, KSA is necessary for the proposed models. Also, many factors must be considered, such as population size, the importance of the area, and regional level size. Since the proposed research's phases require a variety of data depending on where they will be used, intensive efforts have been made to secure these data through communications with official personnel in KSA. Much data already has been collected, such as populations, topographic maps, strategic plans for wastewater systems, wastewater network coverage, water supply, cost functions, etc. Table 1.1 summarizes the relevant data. The research also required reading previous articles, books, official publications of similar neighboring countries, and newspapers.

Element	Description	Key assumptions	Units	Sources	Links
Social	1. Participation and responsibility. 2. Social inclusion.	1. Amount of wastewater from cities - Population - Population growth 2. Water consumption per capita 3. Urban areas 4. Rural areas 3. Wastewater network coverage	m^3/day people % area size	1. General Authority for Statistics (GAS) 2. Ministry of Environment Water and Agriculture (MEWA) 3. World Bank	https://www.stats.gov.sa/ https://www.mewa.gov.sa/ar/Pages/default.aspx https://www.moe.gov.sa/ar/Pages/default.aspx
Environmental	1. Wastewater quality at source. 2. Types of chemical pollution. 3. Wastewater quality standards for the country.	BOD Dissolved oxygen Nitrogen Phosphorous	mg/l	1. MEWA 2. Lecturer review	https://www.mewa.gov.sa/ar/Pages/default.aspx
Technical	1. Performance of system. 2. Flexibility.	- Strategic plan for reuse systems -Capacities and types of WWTP	area size	1. Lecturer review 2. MEWA	https://www.mewa.gov.sa/ar/Pages/default.aspx
Economic	1. Cost functions of - Pipeline - WWTPs - Pumps 2. Affordability.	- Costs subject to decision variables. - Financial limitations	m^3/day dollars or riyals	1. Water Companies 2. Ministry of Economy and Planning 3. Lecturer review	https://www.nwc.com.sa/Arabic/Pages/default.aspx https://www.mep.gov.sa/ar
Candidate locations for Collection and WWTP	1. Locations of candidate collection and WWTPs. 2. Locations of plan WWTPs with capacities.	1. Agricultural 2. Entertainment spots 3. Energy uses	expertise	Google Earth Lecturer review	https://www.google.com/earth/
The topography of Case Study	1. No. of Cities. 2. Distances. 3. Location of cities. 4- Elevations.		area size (km^2)	Google Earth Pro GIS maps Topography Maps	https://www.google.com/earth/ https://momra.gov.sa/

Table 1.1. Summary of the Data Sources.

After all required data for the region was secured, the case study was divided into five different zones for purposes of using treated wastewater. This phase is analyzed wastewater systems against population, average water consumption, observed water demand, growth rate, water supply, losses and infiltration, leakage factors, and wastewater network coverage, using the proposed methodologies developed in phases 1 and 2. The main reason for evaluating the existing plan of the wastewater system is to analyze the effects for most cities in the region. The most problematic region is identified and an alternative solution was proposed in this research. For this phase, it considered two models (M-1, M-2) developed in phases 1 and 2. First, I applied the optimization models to optimizing the layout and locations of WWTPs in order to define the planning network system. Second, I applied the optimization model that considers hydraulic design to minimize the overall cost of the system.

The goal of this phase was not to design a wastewater system that fully satisfies the sustainability objectives, but to test and verify the feasibility of proposed solutions based on optimization methodologies.

Phase 7

The seventh phase is application of M-3 and M-4 to the storm sewer system. M-3 simultaneously determines the layout and the pipe design for storm water systems and is applied to hypothetical scenarios, while M-4 is applied to a real case study. The results of the models' applications were compared with another solution approach. This involved presenting a new approach for pipe design system that can be used in engineering practices.

1.3.3 Objectives of the Models

The objectives of the models that were developed are summarized as follows:

1. Develop an optimization model for layout and locations of a sewer system and WWTP for a regional wastewater system planning, considering hydraulic constraints (M-1). It includes the following sub-objective models:
    a) Minimize total costs of construction and O&M of sewer network that connected from sources to WWTPs.
    b) Minimize total costs of construction and O&M of WWTPs.
2. Develop an optimization model for the hydraulic design of regional wastewater systems.
    a) Minimize the costs of regional wastewater system elements, such as pump stations, commercial diameters, excavation work, and WWTP costs.
3. Develop an optimization model for combined layout and pipe design of storm sewer system. It includes the following sub-objective models:
    a) Minimize total costs of pipelines with commercial diameters and total length of each manhole to outlet manholes.
    b) Minimize total costs of construction of excavation work costs needed to satisfy velocity constraints.
    c) Minimize total costs of hydraulic pipe design for given layouts.
4. Evaluate and investigate the current strategic plan for the wastewater system in Jizan Region, KSA. This evaluation shows tips for the planning system and proposes a new approach to help the strategic plan by considering many cities in the region.

5. An efficient solution method to solve the models. In principle, the models require a large computing effort to be solved. Consequently, another key goal is to develop a solution method that expeditiously provides the right solutions for the models and is efficient even for large and realistic problems.
6. Build scenarios for simultaneous layout and pipe design for the storm sewer system by changing total lengths, design flows, and ground elevations. It identifies the problems in storm sewer systems concerning different types of approaches.
    a) Model for minimizing costs associated with commercial diameters.
    b) Model for minimizing costs associated with manhole depth.
    c) Different length, design flows, and ground elevation for different case scenarios.
    d) Incorporation of different outlet crown elevations.
    e) Evaluate each scenario for layout, hydraulic components, and total cost.

1.4 Organization of the Research

This dissertation contains 10 chapters describing the research development and its results. Chapter 2 is a literature review related to previous models of regional wastewater and storm water systems. The review assures sustainable wastewater and storm water systems, previous optimization models, and water resource planning. The concept of sustainable water resource planning is introduced in detail, including population, climate change, wastewater treatment, and centralized and decentralized systems.

In Chapter 3, the study area and software packages used in this dissertation are defined in detail. The description of the case study issues for regional wastewater and storm water systems are described. Software packages used in this dissertation, such as the general algebraic modeling system (GAMS), Google Earth Pro, and Excel, are presented.

Chapter 4 demonstrates a simple approach to using the INL method to solve a regional wastewater system problem.

Chapter 5 expands and presents a simple approach to using the INL method to solve storm water systems problem.

Chapter 6 demonstrates the application of the model developed in Chapter 4 for planning a regional wastewater system. The model is applied to the Jizan region, KSA.

Chapter 7 describes the model design of a regional wastewater system. The mathematical formulation is described in detail in this chapter. The model is applied to the Jizan region, KSA.

Chapters 8 and 9 demonstrate the model development of simultaneous layout and pipe design of the storm water system. Moreover, they describe the model development for pipe designs for given layouts of the system.

Chapter 10 summarizes this research with conclusions and provides recommendations for future work.

Figure 1.3 shows the flow chart of the dissertation structure and makes explicit the relationships between chapters. Figure 1.4 shows the topography map of the case study area and locations of cities considered for this study.

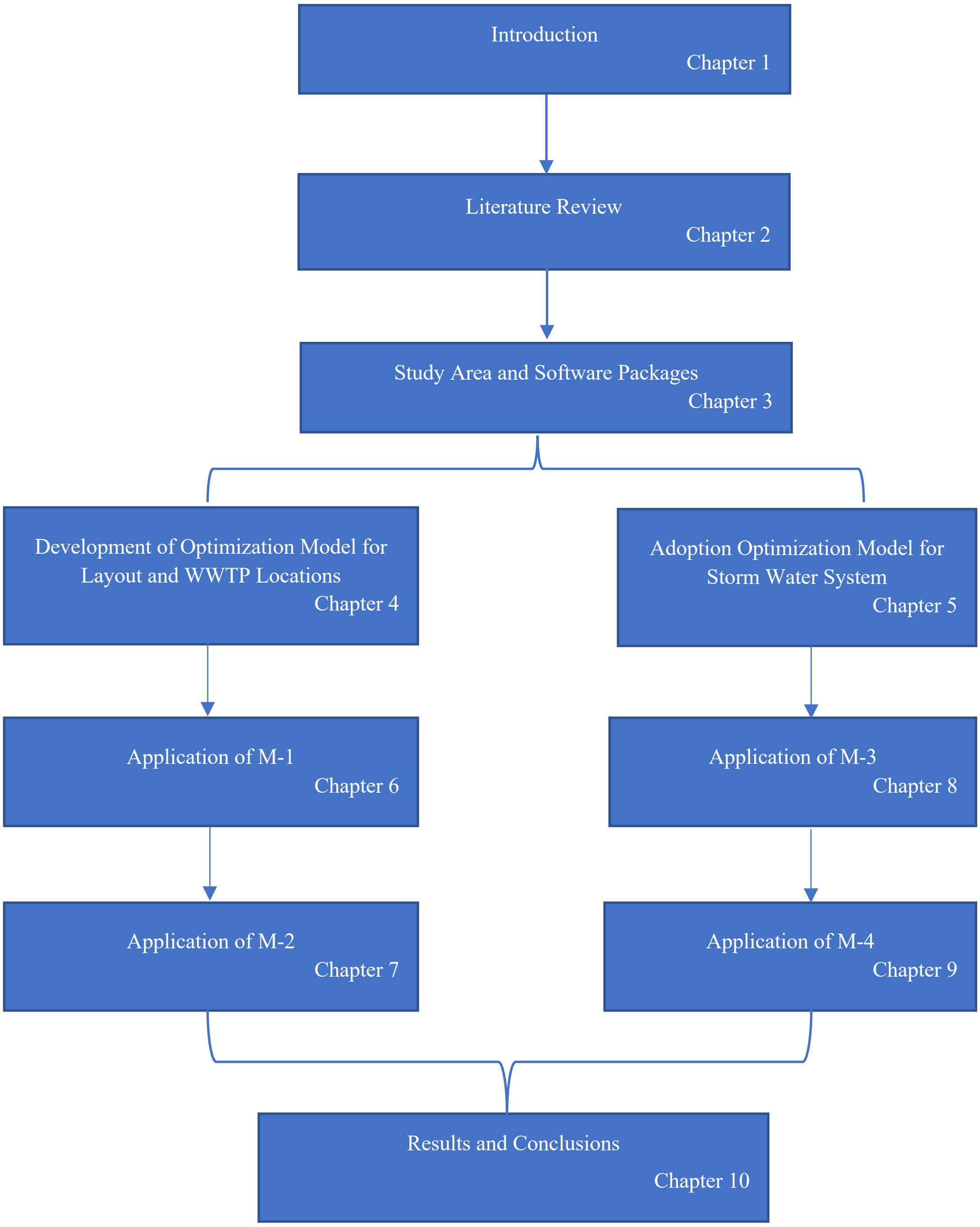


Figure 1.3. Flowchart of Development and Application Phases.

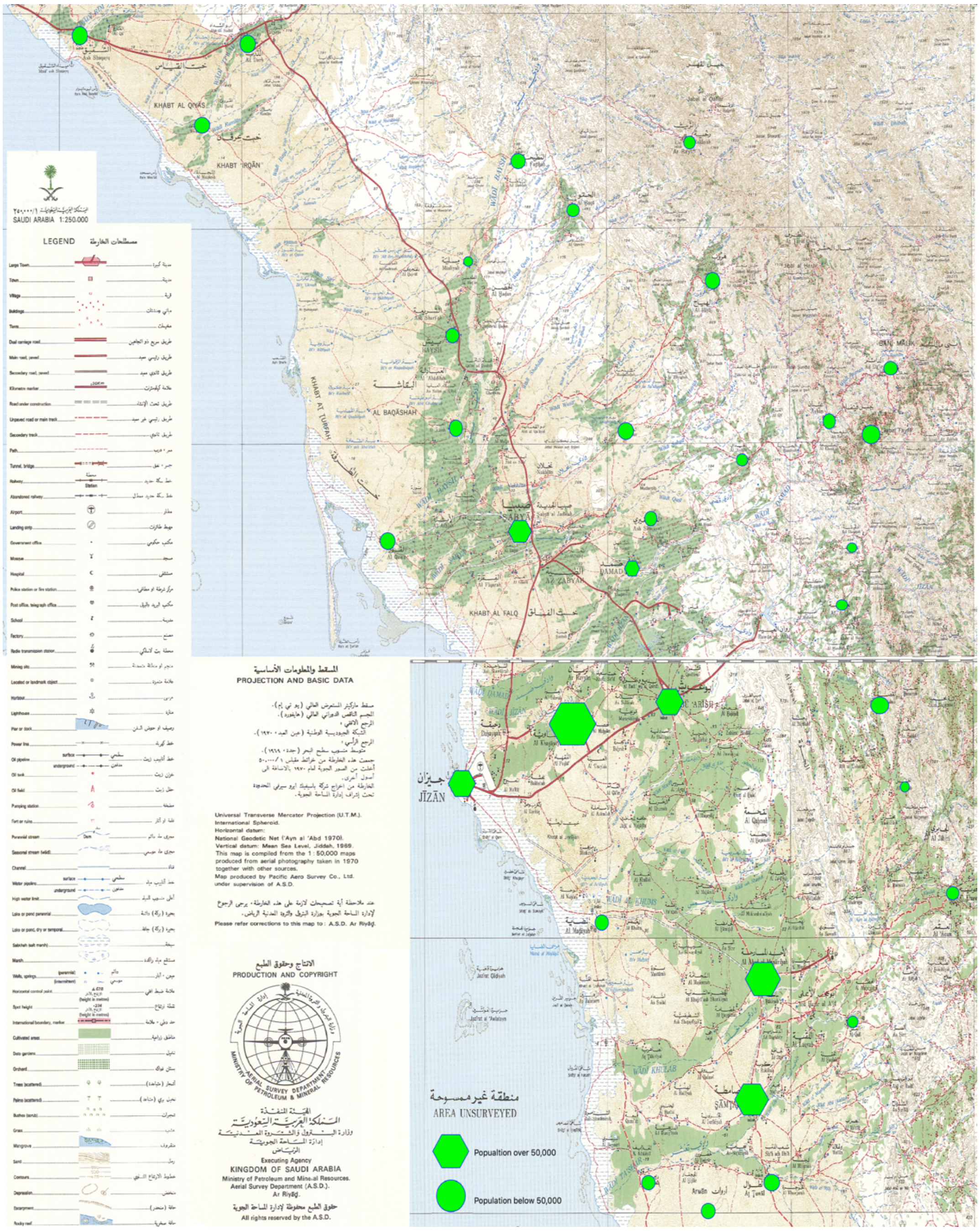


Figure 1.3. Topography Map for Jizan Region, KSA. (Source: Saudi Geological Survey.)

2 LITERATURE REVIEW

2.1 Introduction

One of the most significant natural resources that humans and animals cannot live without is water. In many parts of the world, water is under threat, causing many social, political, and economic problems (HelpSaveNature Staff, 2018). Wastewater and storm water systems planning modeling is one of kind of mathematical optimization model, which has a mathematical function formulated to express some quantity one wishes to optimize (maximize or minimize), subject to a set of technical and environmental constraints. The mathematical function is known as an objective function, which could be single or multi-objective. The objective function and constraints are used to represent realistic environmental conditions in the optimization model. The development of water reuse planning models in the last 30 years reflects simultaneous improvements in programming techniques during the same period.

The regional wastewater treatment systems is a classic optimization problem defined by the transport system and characteristics of the treatment, ensuring quality and minimum costs (Cunha, 2010). The problem can also solve one of these goals:

- To minimize the environmental impact;
- To maximize system reliability;
- To reduce total costs;
- To optimize system flexibility under uncertain conditions;
- To sure equity among users of the system;
- To maximize benefits from reuse of treated effluent.

Problem solving consists of the identification of a system that is composed of many treatment plants, all treating effluent from one or additional polluting sources. The answer should embrace location, type, and requirement standards of treatment plants, similar to the layout of required transport systems. According to de Melo and Camara (1994), the optimization of regional wastewater treatment systems presents many difficulties:

- Objectives are difficult to define accurately.
- Costs functions (which make up the nuclear objective function) are strongly non-linear and limit the application of the most common optimization methods.
- The number of solutions grows exponentially with problem size, creating a need to use optimization techniques.

Wastewater collection networks are the most critical infrastructure of any modern city; they directly influence public health and are essential for environmental protection. Annually, governments spend a lot of money on developing sewer systems, especially in flat areas. However, the associated costs and operational problems could be managed and optimized during the design process. In this regard, the development and application of optimization models to a design of sewer networks are helpful. Via optimization, it is possible to gain a cost-effective design, while systematically meeting all hydraulic and technical constraints associated with the sewer systems. The design of wastewater collection network needs to solve two successive sub-problems: (1) generating the layout and (2) sizing the network's components (Haghighi, 2017). For further information about the optimization of water distribution systems, refer to Mala-Jetmarova Sultanova and Savic (2017).

2.2 Previous Models for Regional Wastewater Systems

Wastewater is considered to be one of the water resources, which could be treated and used for different sources. It is recognized as the primary source of water pollution in urban areas (Cunha et al., 2009). According to the fourth World Water Development Report, over 80% of global wastewater is released into the environment without adequate treatment (UN Water, 2017). For instance, around 80% of KSA's wastewater is still discharged into septic tanks without treatment (Amin et al., 2015). However, many types of water resource problems can be solved by optimization techniques. The optimization approach began to be used for regional wastewater systems planning in the early 1960s.

The literature review on regional wastewater optimization models is divided into three aspects: 1) sewer networks, 2) treatment plants, and 3) the system optimization models, as described by Brand and Ostfeld (2011), de Melo and Camara (1994), and Yen et al. (1976). Several studies have been devoted to the optimization of the water supply system plans for regional wastewater systems, including other components, such as sources for supply water, water treatment plants, and users. Early works include Ocanas and Mays (1981a), in which NLP under steady and dynamic conditions was applied to solve water reuse planning optimization. In their follow-up chapter, the problem was formulated in an active planning model, with single and multiple periods, and solved by successive linear programming (LP) (Ocanas & Mays, 1981b). These studies also considered water quality and nonlinear cost functions for the objective function.

For a detailed survey of the models presented in the literature during this period, see de Melo and Camara (1994). To our knowledge, recent work on optimization models for regional wastewater systems as a combined system (sewer, pumps, and WWTPs) is offered by Zeferino et al. (2012, 2014), who developed robust optimization models by using simulated annealing (SA) for uncertainty of populations and water quality. The models were focused on water quality of the river and flow through the sewer network. Zeferino et al. (2010) developed a multi-objective model to minimize costs and maximize water quality using an SA algorithm for components of sewer pipelines and WWTPs. Zeferino et al. (2017) used a single objective function, either maximizing water quality or minimizing costs for regional wastewater system, and its application is illustrated through a case study in the Una River Basin region, Brazil. Brand and Ostfeld (2011) developed a model focused on developing the cost functions of the regional wastewater system components using a genetic algorithm (GA).

2.3 Previous Models for Storm Water Systems

Optimal design of an urban sewer network requires two considerations: (1) layout design and (2) network component sizing, such as commercial diameters, pipe slopes, and crown elevations. These two optimization problems have different objectives and must be solved completely. The complexity of the problem includes hydraulic constraints, the solution method, and the geography of the study area. The nature of hydraulic constraints, the solution methodology used, and the topography of the study area add further complexities to the design process. The optimization problem is solved by using an MINLP developed in GAMS.

Storm water systems are critical urban infrastructure for flood control. Storm water systems collect street runoff and convey storm water through sewer networks to outlets. Minimization of construction costs for storm water systems is considered to be an objective (Gupta et al., 1983; Karovic & Mays,2014; Steele et al., 2016).

Haghighi (2017) explained three different approaches that have been used to solve the following two problems: 1) full calculation, which generates all layout solutions and designs them hydraulically, and the best design would be chosen Diogo and Graveto (2006), Pereira (1988), 2) separate layout and pipe design, in which the layout can be designed manually and then the pipe network is designed and the problems are disconnected and individually optimized (Afshar, 2010; Bhave, 1983; Haghighi, 2013; Karovic & Mays, 2014; Liebman, 1967; Pan & Kao, 2009; Tekeli & Belkaya, 1986; Walters & Lohbeck, 1993; Walters & Smith, 1995), and 3) simultaneous design, which incorporates layout and pipe design into one optimization model simultaneously and is the only way to reach the global optimum design of a large system (Haghighi & Bakhshipour, 2016; Hsie et al., 2019; Li & Matthew, 1990; Steele et al., 2016). Many of optimization models have been developed for designing storm sewer systems since the latter 1960's. The list of optimization models and solution methods include the following:

- Akfirat and Deininger (1966), Deininger (1966), Deininger (1970), Loucks et al. (1967), Morgan and Coulter (1982), Swamee and Sharma (2013) used LP.
- Dajani et al. (1972), Gidley (1986), Graves et al. (1972), Holland (1966), Price (1978), Smeers et al. (1982) used NLP.
- Botrous et al. (2000), Converse (1972) Klemetson and Grenney (1985), Kulkarni and Khanna (1985), Martin (1980), Walsh and Brown (1973) used dynamic programming (DP).

- Jang (2006), Li and Matthew (1990), Mays (1976), Mays et al. (1976), Mays and Wenzel (1976), Mays and Yen (1975), Yen et al. (1976) used DDDP.
- Dajani and Hasit (1974), Downey and Nakamura (1978), Hsie et al. (2019), Jabbari and Afshar (2002), Joeres et al. (1974), Lejano (2006), Safavi and Geranmehr (2017), Wanielista and Bauer (1972) used linear mixed-integer programming.
- Chiang et al. (1977), de Melo and Camara (1994), McConagha and Converse (1973), Weeter and Belardi (1976) used different types of classic heuristic methods.
- Karovic and Mays (2014), Sousa et al. (2002), Steele et al. (2016), and Wang and Jamieson (2002) used SA.
- Afshar et al. (2006), Cembrowicz (1994), Guo (2005), Hassanli and Dandy (2005), Heaney et al. (1999), Liang et al. (2004), Pan and Kao (2009), Walters and Lohbeck (1993), and Wang and Jamieson (2002) used a GA.

2.4 Previous Models Based on the Iso-nodal Line (INL) Method.

An INL is defined as an imaginary line connecting nodes with the same number of pipes connecting to the outlet of the sewer system, as shown in Figure 2.1. The INLs in Figure 2.1 represent the elevations. Mays (1976) introduced the INL method in the 1970s to describe dendritic flow networks, such as storm and sanitary networks. Mays and Wenzel (1976) applied the concept to determining a minimum cost design of a multilevel branching storm sewer system using DDDP. Mays (1976) and Yen et al. (1976) used the INL method for a collection type system (storm sewer system) by computing the minimum cost layout and design of sewer systems for arbitrary topographic, physiographic, and hydrologic conditions. Tekeli and Belkaya (1986) provided the fundamental concepts for solving the layout problem of a sewer system and developed a model utilizing the DP method and the concept of iso-drainage lines. Bennett and Mays (1985) extended Mays and Bedient's (1983) model to optimize individual basin outlet structures and downstream channel reaches. This model was modified and applied successfully in the field by Taur et al. (1987) to the Walnut Creek watershed in Austin, Texas. More recently, Steele et al. (2016) used the same concept for layout and pipe design of a sewer storm system by interfacing two optimization models: MINLP and SA programming, developed in GAMS and Excel. This improved methodology was applied in Chapter 8 to solve a problem in one optimization model.

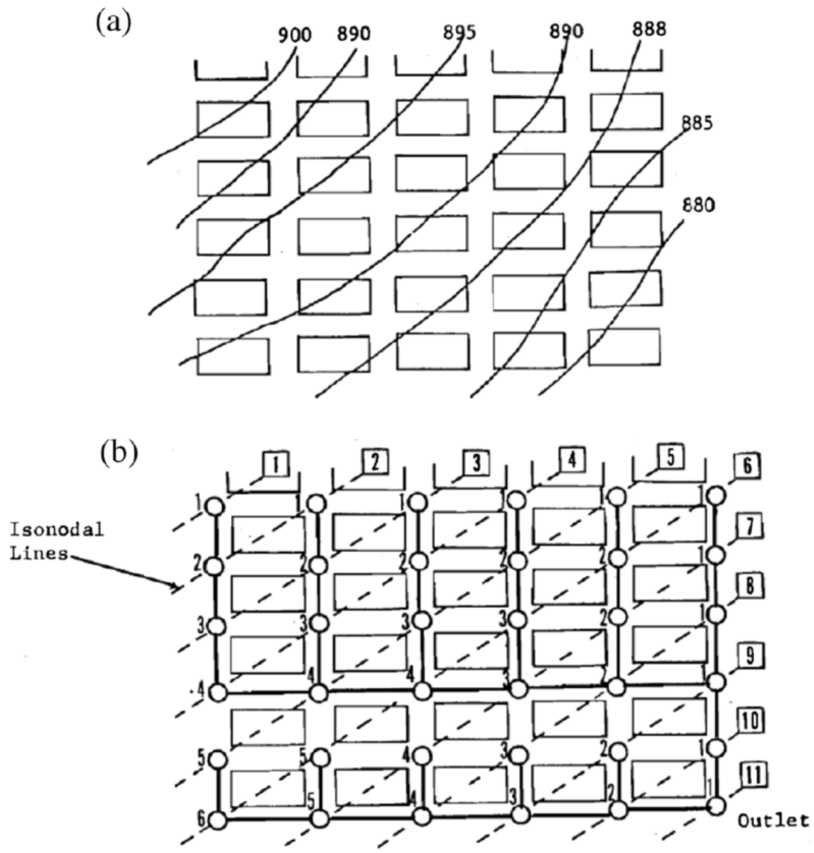


Figure 2.1. Iso-nodal Line Method, Including a) Street Layout and Surface Elevation Contours and b) Layout and Iso-nodal Lines (Mays, 1976).

2.5 Sewer Network Design

The term *network* has a special meaning in engineering and science. For any dendritic tree networks, there are nodes connected by links to one or more outlet for any purpose. In the collection network, the nodes are collection/sources or waste/water treatment plants, and links are sewer pipelines. Wastewater system flows are horizontally conveyed by sewers toward the WWTPs and vertically collected by secondary drains to collection nodes. The layout of the system depends on outlet location, problem size, and topography of the area. The best layout is the first step in designing a new wastewater system that has too many alternatives, which are supposed to collect sewage flow gravitationally. The designer, therefore, depends on the topography of the region as a fundamental rule to follow natural ground slopes towards the network outlet. A nearly optimum layout can be obtained depending on the designer's experiences and the steepness of the area in which sewers must comply with the natural ground. In other words, a cost-effective layout can be designed for any steep area, depending on engineering experience. Due to suitable natural slopes, this design reduces the diameter sizes and excavation work for the sewers and the need for pumping stations, which reduce the total costs of the system's construction. The problem is different and challenging to solve for flat areas where there is no significant change in topography elevations. Thus, the optimizer cannot make a decision on a unique layout based on natural ground slopes. There are many possible locations of sewer connectivity and WWTP of the network. In flat areas, engineering decisions and experiences are not sufficient to design the sewer layout of flat regions, where the number of layouts increases exponentially with the number of sewers.

Because the network runs on natural slopes, its design specifications and construction and operational costs are sensitive to the configuration of the layout.

As a result, the importance of layout selection of urban drainage systems is impacted by hydraulic factors, such as flow rate, sizes and gradients of pipes, and the effect of pumping stations (Li & Matthew, 1990). Using optimization techniques, particularly in flat regions where standard approaches are not very effective, is useful and essential. The layout sub-problem belongs, as described, to a difficult class of combinatorial mathematics, which requires some background understanding of graph theory (Haghighi, 2017).

2.6 Sustainable Development and Water Resources Sustainability

2.6.1 Population

According to UN (2017), the global population is expected to increase: around 2.5 billion more people will live on earth by 2050, and approximately 85% of this population is likely to be in developing countries. Currently, 84% of the worlds' population living in developing countries, and this is expected to increase to 88%. On the other hand, developed countries already have slower growth rates because of their stable birth rates and increasing death rates, due to an aging population. Developed countries, such as Italy, Germany, Japan and Spain, face decreasing rates in the population. Already there are limited amounts of economic and natural resources available in developing countries for the current population, which is expected to be less per capita in the future with the predicted population increase. Furthermore, according to UN (2018), 90% of the anticipated rise in the world's urban population will occur in the urban regions of Asia and Africa between now and

2050. In addition, the projected urban growth will be concentrated in cities in the developing world, where the correlation between the rate of urbanization and economic growth has been weaker. Globally, more people live in urban areas than in rural areas, with 55% of the world's population residing in urban areas in 2018. In 1950, 30% of the world's population was urban, and by 2050, urban areas are expected to account for 68% of the world's population. The increasing population in urban areas will bring problems of infrastructure, housing, declining sanitation, environmental pollution, and an inadequate water supply, especially in developing countries (UN, 2009).

2.6.2 Climate Change

While many previous studies have looked at the global changes and impacts of climate change and related variability on water resources, few have focused on an assessment of the specific effects and needed adaptation and mitigation for water systems in cities across the globe. Climate change distresses water resources by changing water storage patterns through the hydrological cycle. Changing temperatures result in shorter spring snowmelt and increasing winter runoff, therefore wholly evolving the overall seasonal stream flow pattern . According to James and O'Neill (2010) climate change might affect water, wastewater, and storm water infrastructure and provided recommendations to assist engineers and decision-makers in addressing these impacts.

UNESCO (2017) reported the effects of climate change on water resources sustainability and management, especially in the case of water/wastewater systems. One challenge that water managers face due to climate change is that long-term plans and water/wastewater system designs can no longer be based on historical data due to extreme differences in seasonal patterns. Climate change affects economic growth because of the uncertainty and unpredictability of long-term investments, specifically infrastructure design.

Researchers have developed several models and performed studies into how climate change affects wastewater systems. For example, Zouboulis and Tolkou (2015) studied climate change impacts on WWTPs' performance using two factors. Processes in a WWTP are impacted by climate change, where more extreme weather events and the earlier runoff of snowmelt lead to more untreated sewer overflows and enhanced flooding, etc. The second factor involves wastewater treatment: greenhouse gases, methane (CH4), and nitrous oxide associated with nitrification/denitrification processes, as an intermediate product, can be released into the atmosphere. Because of a lack of water resources, wastewater reuse will become more necessary as climate change accelerates. Plósz et al. (2009) investigated climate change effects on combined sewer systems receiving sewage collected during winter operation in Norway. The results show that the WWTP influent flow rates are significantly increased during temporary snow melting periods. Raje and Mujumdar (2010) investigated the impacts of climate change on reservoir performance on the Mahanadi River in Orissa, India and identified reliability, resiliency, vulnerability, and deficit ratio of hydropower s the four performance criteria. A General Circulation Model used the evolution of hydrological scenarios and climate change adaptation policy for reservoir operation using DP. The hydropower generation reliability decreased in most scenarios in this study.

Jyrkama and Sykes (2007) studied the temporal and spatial changes of groundwater recharge to climate change. The study used a simulation of the hydrologic cycle for 40 years of weather data in the Grand River watershed. The results showed that climate change would lead to increased groundwater recharge. The higher intensity and frequency of precipitation will also contribute significantly to surface runoff, while global warming may result in increased evapotranspiration rates. The study concluded that climate change will result in important spatial changing. Ficklin et al. (2009) used a climate change sensitivity assessment of a highly agricultural watershed using the Soil and Water Assessment (SWAT). SWAT was used to model the hydrology and impact of climate change in the agricultural San Joaquin watershed in California. The results of this study showed that atmospheric carbon dioxide (CO_2), temperature, and precipitation changes have serious effects on water yield, evapotranspiration, irrigation water use, and stream flow. It also indicated that the hydrology of the San Joaquin watershed is linked to climate changes. Climate change has a serious effect on water quality and quantity, which will continue to change accordingly.

Trenberth (2011) emphasized that there is a direct influence of global warming on the hydrological cycle, leading to extreme weather conditions. For example, increased heating leads to greater evaporation and surface drying, thus increasing the intensity and timing of drought. The study recommended storing water during floods and using it during droughts. A total of 70% of all water consumed in the Phoenix area is used outdoors, much of it to irrigate urban landscapes for cooling weather purpose, reducing the urban heat island effect, and reducing energy demand. According to Guhathakurta and Gober (2007), decreased precipitation and increased temperature and droughts have led to increased water consumption per capita in the region. Chowdhury et al. (2016) investigated the effects of climate change on crop water requirements in Al-Jouf, KSA, which have had a negative impact and required better planning for water resources management. House-Peters and Chang (2011) studied the climate change effects on residential water consumption in suburban Hillsboro, Oregon. Based on the model that they developed, an increase of 3°C in August would increase water consumption by 4061 L per household. Water stress has increased due to climate change in some parts of the world. According to Jenerette and Larsen (2006), climate change has various impacts on water resources, such as reduced water supply availability. Olmstead (2014) suggested to the water institutions and decision-makers policies for managing water demand, considering climate change impacts. They provided some solutions, such as water-conserving technologies and mandatory water use restrictions, to limit water use. These solutions would help secure the water supply and decrease the impacts of climate change. DeNicola et al. (2015) studied climate change and water scarcity in KSA as a case study, providing an illustrative example of climate change and water scarcity issues. First, it is an example of how the climate and unsustainable human activity go hand-in-hand in creating stress on and depleting water resources. Second, as both a water-scarce nation and a very wealthy one, KSA also serves as a leading example for adaptation and mitigation, which can be very costly. Finally, recent reports state that KSA is beginning to convert many conventional energy sources to renewable and more sustainable ones (i.e., solar) (Carrington, 2015; Clark, 2015).

2.6.3 Wastewater Treatment

Wastewater treatment is a combination of physical, chemical, and biological processes to remove wastewater constituents. Physical processes remove the removal of substances by the use of gravity (i.e., natural forces). Physical barriers, such as filters and membranes or ultraviolet (UV), are used for disinfection. The use of membranes is increasing because of the high quality of flow after treatment and the forceful removal of waste contaminated (Liu et al., 2009). Membrane systems have high energy consumption and levels of operation and maintenance (Visvanathan et al., 2000). Chemical processes are used for disinfection and heavy metals removal. For example, polyelectrolytes can remove solids and BOD, but it is difficult to treat and dispose of the sludge generated, so chemicals help primary treatment (UN Water, 2015). Chemically advanced oxidation can be used to remove endocrine-disrupting compounds. Biological processes in wastewater treatment imitate the degradation that typically occurs in rivers, lakes, and streams. To enhance the removal of pollutants and stabilization of sludge, biological processes are used in WWTPs, which are engineered to boost biochemical degradation under carefully controlled conditions. The biological processes in the bioreactors can be aerobic or anaerobic.

The former often needs more energy to maintain the aerobic conditions inside the reactor and convert the organic waste to biomass (sludge) and CO_2. However, it stops the formation of CH_4, which has a more significant effect on climate warming than CO_2 (Cakir & Stenstrom, 2005). Anaerobic treatment processes generally require less energy and have a lower sludge production and generate CH4, but this can be captured and used as an energy source. These three processes (physical, chemical, and biological) are combined to achieve different levels of wastewater: preliminary, primary, secondary, tertiary, and quaternary. The kind of components, pollution load, anticipated use of treated wastewater, and economic affordability are the main factors in selecting the appropriate technologies. UN Water (2015) provided some examples of technologies, the kind of sewage they are generally used for, and their advantages and disadvantages. Sewage sludge is one wastewater therapy by-product. The sludge generated is usually rich in nutrients and organic matter, which is an appropriate soil conditioner and fertilizer material.

Unfortunately, the benefit value of sewage sludge is not realized due to the focus on pathogens, heavy metals, and other compounds that it may contain. Other useful by-products from wastewater include biogas (i.e., CH_4) and heat, which can be treated for advantageous use either in the treatment plant or the community.

The actual management and operation of wastewater treatment systems is complex; a risk assessment approach can evaluate the series of components that make up the system. These assessments can help ensure their proper functioning under expected levels of efficiency and highlight weak links in the series that could cause health and safety issues.

2.6.4 Centralized and Decentralized Wastewater Systems

Centralized wastewater management approaches have been ineffective, as current traditional lagoons or WWTPs with activated sludge do not operate or operate far below capability. The centralized and complex technology-based solution to wastewater management was noted to have failed in addressing the wastewater problem in Nepal. The primary causes of failure are the high capital investment, high operational and maintenance costs, complexity of the systems, lack of spare parts, and lack of skilled resource persons (Jha & Bajracharya, 2014).

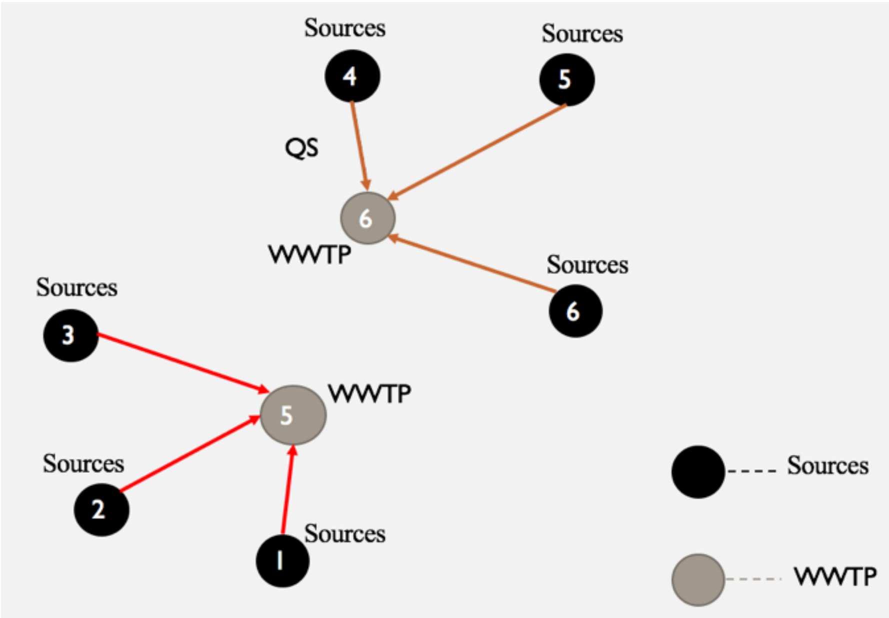
A decentralized concept cannot be the solution to all wastewater management problems in specific cases, where centralized treatment plants are more appropriate. Many potential benefits of decentralized strategy indicate that it deserves greater attention, especially in smaller communities and urban fringe. No work has been performed on finding a balance between centralized and decentralized systems (Ahluwalia, 2012).

The U.S. Environmental Protection Agency (EPA, 2005) reported that decentralized wastewater treatment can be a smart alternative for communities considering new systems or modifying, replacing, or expanding existing wastewater treatment systems. For many cities, decentralized treatment can have the following properties:

- Cost-effective and economical:
  - Avoids large capital costs
  - Reduces operation and maintenance costs
  - Promotes business and job opportunities
- Green and sustainable:
  - Benefits water quality and availability
  - Uses energy and land wisely
  - Responds to growth while preserving green space
- Safe while protecting the environment:
  - Protects public health and water quality
  - Protects community health
  - Reduces conventional pollutants, nutrients, and emerging contaminants
  - Mitigates sewage contamination and health hazards

The relative importance was evaluated under two management/design conditions: (1) centralized versus decentralized wastewater treatment and (2) decentralized wastewater plant location (Hwang et al., 2014). Figure 2.2 shows the difference between centralized and decentralized systems. The QS is wastewater flowing from sources (1,2,3,…6).

a)



b)

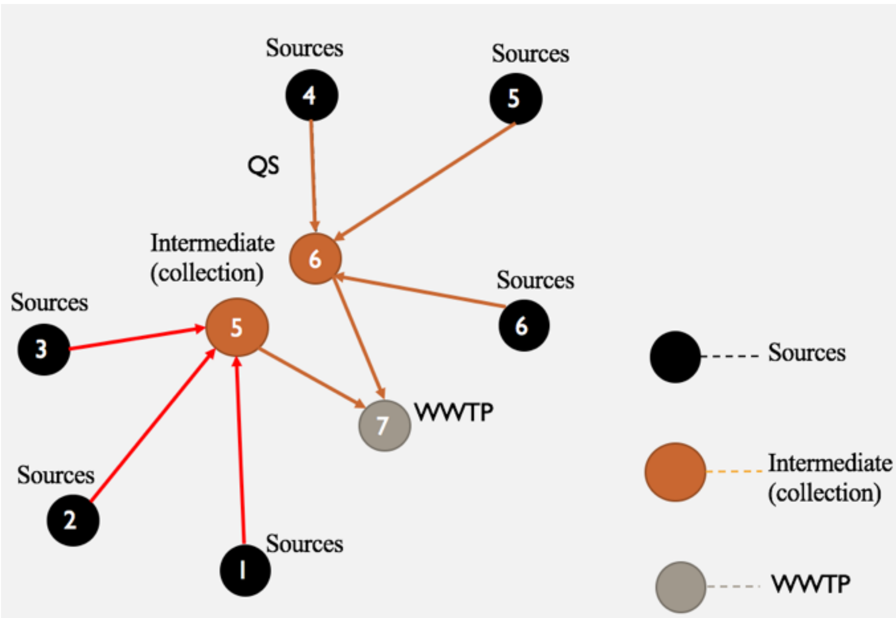


Figure 2.2. a) Decentralized Wastewater Systems and b) Centralized Wastewater Systems

3 STUDY AREA AND SOFTWARE PACKAGES

3.1 Introduction

This chapter provides detailed information about the study area in KSA and the software packages used in this study. The developed wastewater system is located in the Jizan Region, KSA. Data were collected through many sources, such as MEWA, the General Authority of Statistics (GAS), the Ministry of Economy and Planning, the Saudi Geological Survey, etc. This chapter contains information about the population, flow rates, and geomorphologic features of the study area, as well as data processing techniques that are utilized in this study.

3.2 Software Packages

In this study, software packages were used to evaluate and develop optimizations models of regional wastewater and storm sewer systems. The software packages listed below were mainly utilized to analyze the quality of the optimization models of the systems and the data collection for a case study.

3.2.1 GAMS Software

GAMS software is a mathematical programming and optimization tool for modeling small to large scale and simple to complex applications. In the early 1980s, there was a focus on the development of modeling systems for the analysis of sizeable mathematical programming problems. One of the first of these was GAMS, which merges ideas from mathematical programming and electronic database theory and is supposed to handle strategic modelers (GAMS, 2017; McCarl et al., 2012). GAMS includes the following features:

- Provides a high-level language for large and complex models.
- Allows changes to be created in model specifications easily and directly.
- Allows the unambiguous statement of algebraic relationships.
- Provides an environment where model development is facilitated by subscript-based expandability, allowing the modeler to begin with a small dataset; after verifying correctness, it can expand to a much broader context.
- Is inherently self-documenting, allowing the use of more extended variables, equations, and index names, as well as comments, data definitions, etc. GAMS is designed so that the model structure, assumptions, and any calculation procedures used in the report writing can be documented as a byproduct of the modeling exercise in a self-contained file.
- Is an open system, facilitating interface with the newest and best solvers, while being solver-independent, allowing different solvers to be used on any given problem.
- Automates the modeling process, including:
    - Data calculation;
    - Verifying algebraic model statements;
    - Checking formulations for apparent flaws;
    - Interfacing with a solver;
    - Saving and submitting an advanced basis when using related solutions;
    - Permitting usages of the solution for report writing.
- Permits portability of model formulation between computer systems, allowing usage on a variety of computers, ranging from PCs to workstations to supercomputers.

- Facilitates a simple change in solution methodology (solver selection).
- Promotes the import and export of data to and from other computer packages.
- Allows use by groups of varying expertise.
- Provides example models that may assist modelers through the provision of a model library.

GAMS can solve many water resource problems using different solution methods (Aljanabi et al., 2018b; Mounir et al., 2019; Oxley et al., 2016). GAMS was used to develop optimization models for designing regional wastewater treatment and storm sewer systems. The mathematical formulations developed using GAMS are MINLP and NLP. The primary focus is on MINLP.

3.2.1.1 MINLP in GAMS

Mathematically, the MINLP problem could be stated as follows:

Maximize or Minimize

$$A(x) + B(y) \quad (1)$$

subject to

$$D(x) + E(y)\ \Omega\ 0$$

$$L < x < U$$

$$y = \{0,1,2,..\}$$

Where x is a vector of variables which are continuous real numbers, A(x) + B(y) is the objective function, D(x) + E(y) represents the set of constraints, Ω is some mixture of ≤, = and ≥ ,U and L are the upper and lower bounds vectors of the variables, and y is a vector of variables that can only take integer values, {0,1,2,..} (GAMS, 2017).

GAMS developed for MINLP generally follows two approaches:

- Outer Approximation/Generalized Bender's Decomposition: These algorithms alternate between solving a mixed-integer LP master problem and NLP sub-problems. For example, the DIscrete and Continuous OPTimizer (DIOCPt) solver.
- Branch-and-Bound: Branch-and-bound methods for mixed-integer LP can be extended to MINLP with many tricks added to improve their performance. For example, Branch-And-Reduce Optimization Navigator (BARON) solver.

An outer approximation is a fundamental approach for solving MINLP models, as suggested by Duran and Grossmann (1986). Based on principles of decomposition, outer-approximation, and relaxation, the proposed algorithm exploits the structure of the original problems. The new issues consist of solving an alternating finite sequence of NLP sub-problems and relaxed versions of a mixed-integer linear master program. The flow chart for outer approximation is given in Figure 3.1.

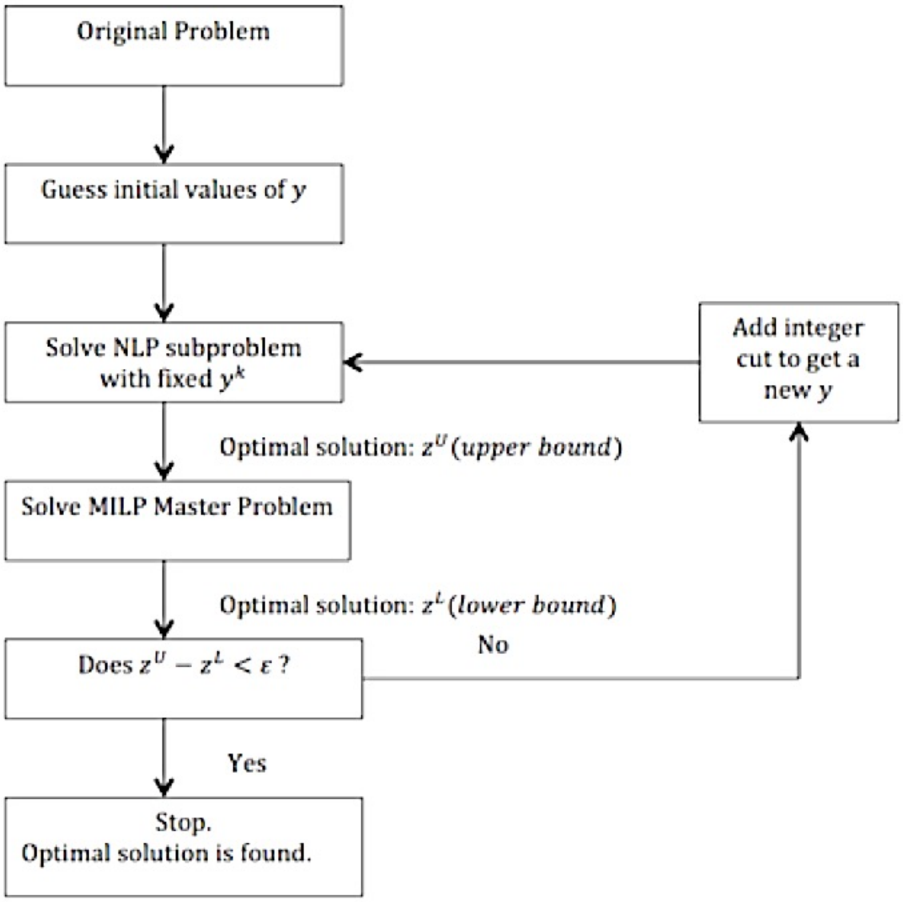


Figure 3.1. Flowchart of the Outer Approximation Algorithm. (Source: Northwestern.edu.)

3.2.1.2 Global and Local Optimum Solution of MINLP

A number of solvers for mathematical programming models have been hooked up to GAMS. A brief description of each solver with the model types and platforms supported by each solver is given. You can choose your default solvers using the Integrated Development Environment (IDE). DICOPT and BARON solvers are discussed here, which provide a global and local solution for the MINLP problem.

DICOPT solves MINLP model types developed by Viswanathan and Grossmann (2002). The models explained many variables, such as linear binary or integer variables and linear and nonlinear continuous variables. The algorithm does not necessarily obtain the global optimum. DICOPT implements extensions of the outer-approximation algorithm for the equality relaxation strategy. Moreover, the DICOPT approach solves NLP and Mixed Integer Programming (MIP) problems. That allows the researcher to match the best algorithms to the issue at hand and guarantees that enhancements in the NLP and MIP solvers are exploited. Licenses are required for such solvers. DICOPT solves MINLP model types. In NLP, it is crucial that the modeler help the solver by specifying as narrow range as possible between the lower and upper bounds. It is also beneficial to define an initial solution from which the solver can start searching for the optimum. When using DICOPT and CBC, it is critical to have the constraints represent to the fullest extent possible the link between continuous and integer variables.

BARON is a GAMS solver for NLP and MINLP problems for the global solutions. “While traditional NLP and MINLP algorithms are guaranteed to converge only under certain convexity assumptions, BARON implements deterministic global optimization algorithms of the branch-and-bound type that are guaranteed to provide global optima under fairly general assumptions. These include the existence of finite lower and upper bounds on nonlinear expressions in the NLP or MINLP problem to be solved” GAMS (2015). BARON is eligible for solving models of the following types: LP, MIP, Relaxed Mixed Integer Program, NLP, Discontinuous Nonlinear Program, Relaxed Mixed Integer Nonlinear Program (RMINLP), and MINLP. If

BARON does not specify the default solver for these models, the following command can be invoked before the solve statement:

Option (Model type) = baron;

For more information about GAMS, refer to the GAMS documentation, which has much valuable information (https://www.gams.com/latest/docs/).

3.2.3 Microsoft Excel

Microsoft Excel offers basic spreadsheet operations and data manipulation. In this study, Excel spreadsheets for wastewater analysis were exported to the GAMS software optimization model. Specifically, wastewater discharges calculations of source nodes or cities were conducted in Excel. Excel was also used for data analysis, such as quantity and average consumption per capita for regions in KSA at certain times. Running GAMS in the background of Excel is also possible. However, it is challenging, requiring programming effort in Visual Basic for Applications (VBA), the language behind GAMS and Microsoft Office products like Excel. In GAMS' user guide, there is an example related to a transportation problem in which GAMS is called from Excel and solves the problem in the background.

3.2.4 Google Earth Pro

Google Earth Pro lets the user view anywhere on earth using satellite imagery, maps, terrain, and 3D buildings, which is very helpful. The user can explore rich geographical content, save toured places, and share locations with others. In this study, distances, topography, and locations of the case study can be discovered through this program.

3.3 Water Issues in KSA

There are many conflicts and crises in many regions around the world. Water reuse is one of these problems, and many areas, unfortunately, do not take it into account, especially in arid and semi-arid regions. In this study, M-1 and M-2 were applied to a real-world problem to provide a sustainable solution for humankind. The location of the case study was the Jizan Region, KSA. KSA's capital city, Riyadh, has a population of approximately 32.28 million and a growth rate of 2.2% (World Bank, 2016). As shown in Figure 3.2, KSA is bordered by Kuwait, Iraq, and Jordan to the north, the Arabian Gulf, United Arab Emirates, and Qatar to the east, Oman and Yemen to the south, and Egypt and the Red Sea to the west. As shown in Figure 3.3, KSA is divided into 13 areas, called emirates (Saudi Geological Survey, 2012), and each emirate is divided into many regions.

Recently, the government of KSA started applying new water tariffs on formerly subsidized water, due to the increasing cost of debt and the decline of oil revenues (Ouda, 2013). This solution is the government's first step to address water shortages. The water tariff comes as warnings that KSA's groundwater will run out in the next 13 years. The government estimated a decrease in water levels in agricultural regions, indicating the seriousness of the situation. This is an unsafe situation for all future farms that depend on these aquifers (Independent, 2016). Experts believe that the water crisis derives from the decision in KSA to grow wheat in 1983 CE (Independent, 2016). Wheat farming is now banned, but the cultivation of hay, olive trees, and date palms still continues.

Figure 3.2. Map of KSA. (Source: University of Texas at Austin).

Figure 3.3. Emirates (regions) in KSA. (Source: University of Texas at Austin).

The UAE has invested in cloud-busting technology that shoots flares containing table salt into the clouds to induce more rain (Independent.com, 2016). KSA instead relies on two sources of water: groundwater and desalinated water, which involves removing salt from seawater. The desalination process is hugely energy intensive, releases harmful gases to the environment, and is costly. A National Geographic (2017) report stated that, 40 years ago, when intensive modern farming started, there were a staggering 120 cubic miles (500 cubic kilometers) of groundwater in the Saudi desert, enough to fill Lake Erie. But in recent years, five cubic miles (21 km^3) are pumped to the surface for use in agriculture annually. None is replaced by rain, because rain is scarce in the area. Sources of water within the KSA is around 1% compared with average global water resources (DeNicola et al., 2015).

Wastewater collection and treatment systems in KSA cover approximately 49% of the urban areas; approximately 25% of the treated wastewater is used for landscaping and crop irrigation (MEWA, 2017). Currently, the government aims to provide 100% sewage collection, treatment services, and treated wastewater to every city with a population above 5000 by 2025 (Drewes et al., 2012). Although agriculture is the largest water user, most wastewater is generated outside this sector (Qadir et al., 2010). Agriculture accounts for more than 75% of total water usage in the Middle East. With growing demand due to population growth and greater living standards, however, water will be reallocated to the domestic and industrial sectors (Ouda, 2014a). For WWTPs, the most commonly used secondary treatment technology in KSA is conventional activated-sludge systems. Many new plants are currently being built, mainly in cities managed by the National Water Company (www.nwc.com.sa), including Riyadh, Jeddah, Makkah, Medinah, and Al Taif. Wastewater flow in the six largest cities (Riyadh, Jeddah, Makkah, Al Taif, Medinah, and Dammam) receives tertiary or secondary treatment levels. According to King Abdullah University of Science and Technology (KAUST), sewer systems cover only 55% of Riyadh, 50% of Jeddah, 45% of Makkah, 50% of Al Taif, 68% of Medinah, and 78% of Dammam. For more information about the recent wastewater situation in KSA, see Al-Zahrani et al. (2016), Amin et al. (2015), Bulletin (2016), Drewes et al. (2012), and Ouda, (2014b, 2015, 2016). However, none of these articles discussed optimization models that could be used in the region to solve the problem.

As shown in Figure 3.4, the Jizan Region is located in the southweast part of KSA. It is divided into 23 regions and has a total population of approximately 1.5 million people. This study focuses on the Jizan Region, KSA because it has poor wastewater service, as shown in the figures and statistics in Appendix A, and because the average water use has been increasing annually.

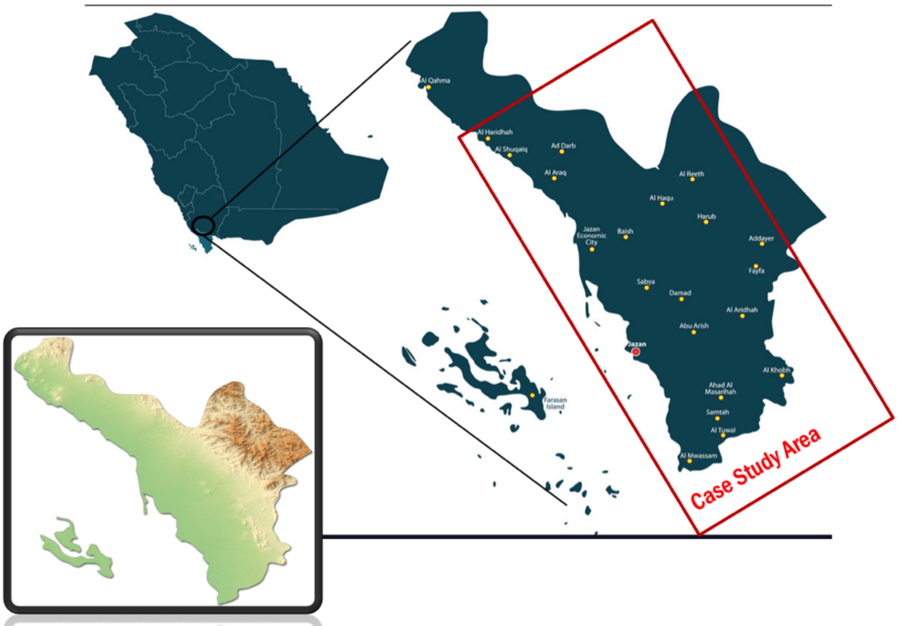


Figure 3.4. Jizan Region Divided into 23 Regions. (Source: Colourbox.Com.)

As shown in Figure 3.4, the topography of the Jizan Region gradually rises toward the east. Adding pumps to the system increases the excavation costs; if the excavation costs exceed the limit, pump station parameters should be intrduced to the model, as described in M-2. The elevation in the eastern region is 700 ft from sea level. Jazan city is the capital of the region. Table 3.1 shows existing and projected wastewater flow in the Jizan Region, which has only one WWTP, with a capacity of 20,000 m^3/day and a planned capacity of 112,000 m^3/day, an 85% increase. The need for treated wastewater systems provided the motivation to discover new ideas and solutions that can be applied to real-world problems.

Table 3.1. Existing and Projected Wastewater Flow in Jizan Region, KSA (adapted from Ouda, 2016).

Region	Calculated wastewater flow (1000 m^3/day)			Wastewater treatment plants: number and capacity (1000 m^3/day)						
				Existing		Under construction		Planned		Total planned future
	2010	2025	2035	No.	Capacity	No.	Capacity	No.	Capacity	Capacity
Jizan	203	301	381	1	20	0	0	22	112	132

Groundwater in the Gulf is running out because it has the highest levels of water consumption per capita in the world. The Saudi newspaper, *Al-Watan Arabic Daily*, reported that daily water use per capita is 265 liters in KSA, double the EU average (Biswas & Tortajada, 2017; GAS, 2016; Third World Center for Water Management Organization, 2017). See Figure 3.5.

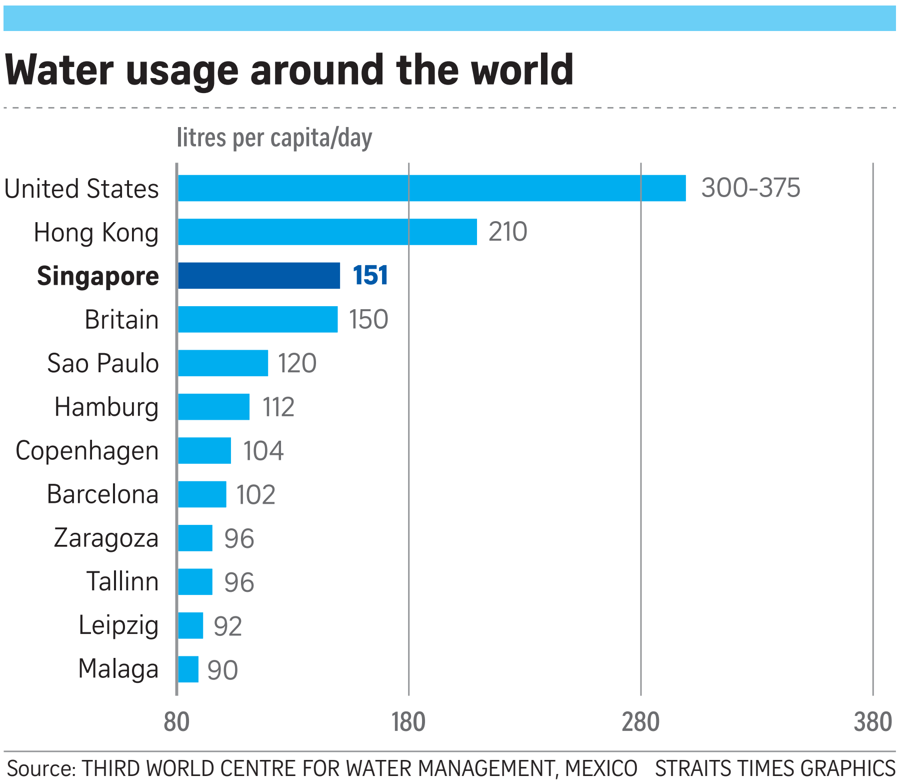


Figure 3.5. Average Daily Water Usage Per capita Around the World. Source: Third World Center for Water Management (2017).

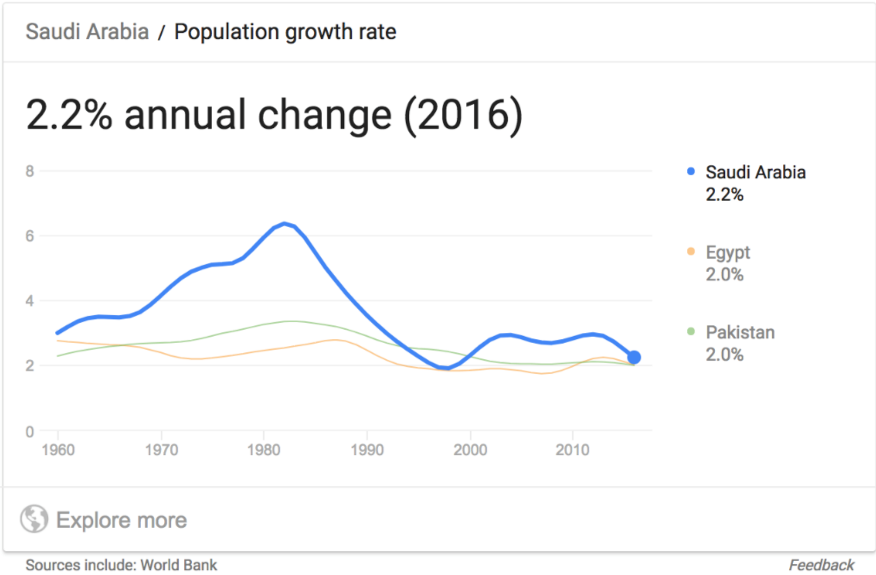
The studied system is a regional wastewater system that contains wastewater, pipelines, and WWTPs. This water collection system collects wastewater from sources through sewer network that joins wastewater plants. The optimization model is one action that can be used in decision models (Schlüter et al., 2017). Applying these models to the real world would be more sensitive because, when there is a change in one of the constraints or parameters, it changes the layout and/or design of the system.

3.3.1 Population and Growth Rate

The population is increasing under uncertain conditions. Relinquishing reproduction is important to note. No one method can safeguard us from overpopulation. Freedom to reproduce will demolish us all (Hardin, 1968). Here, we consider future population growth; planning will cover the uncertainty of growth. KSA has had a growth rate between 1.9% and 6.2% in the past 50 years (World Bank, 2016). See Figure 3.6.

This case study includes 34 cities with a total population of 1.3 million people (GAS, 2017). These cities are located in the Jizan Region, KSA and are surrounded by 9500 km^2 of agricultural lands, as shown in Figures 3.7 and 3.8. The population, location, and elevations of cities are described in Table 3.2. The candidate locations of collection and WWTPs are in populated and agricultural areas. However, they would likely be at lower elevations of source nodes. Chapters 6 and 7 provide more detailed explanation about these cities and locations.

a)



b)

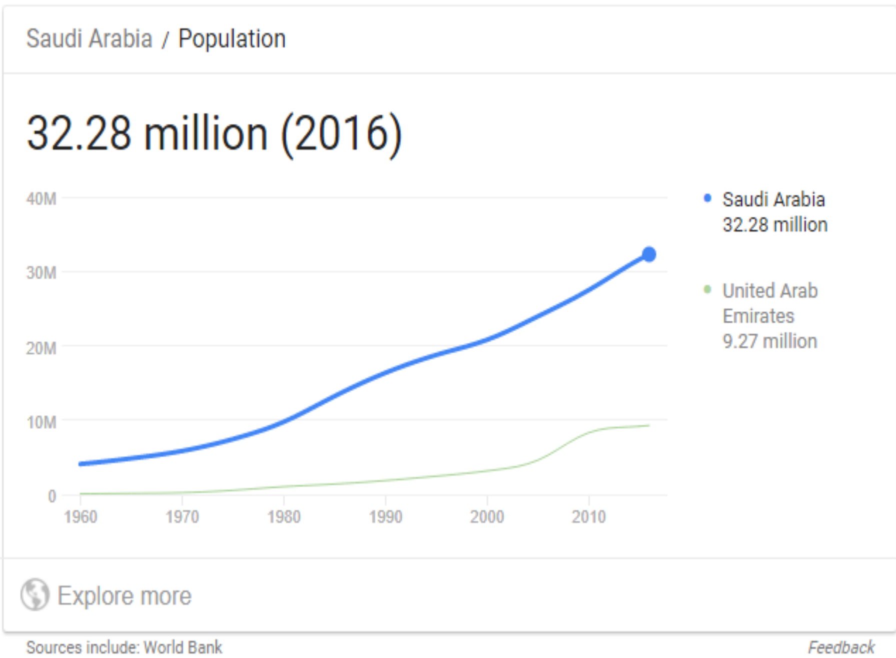


Figure 3.6. a) Annual Growth Rate of Population KSA from 1960-2016, and b) Population of KSA from 1960-2016. (Source: World Bank, 2016.)

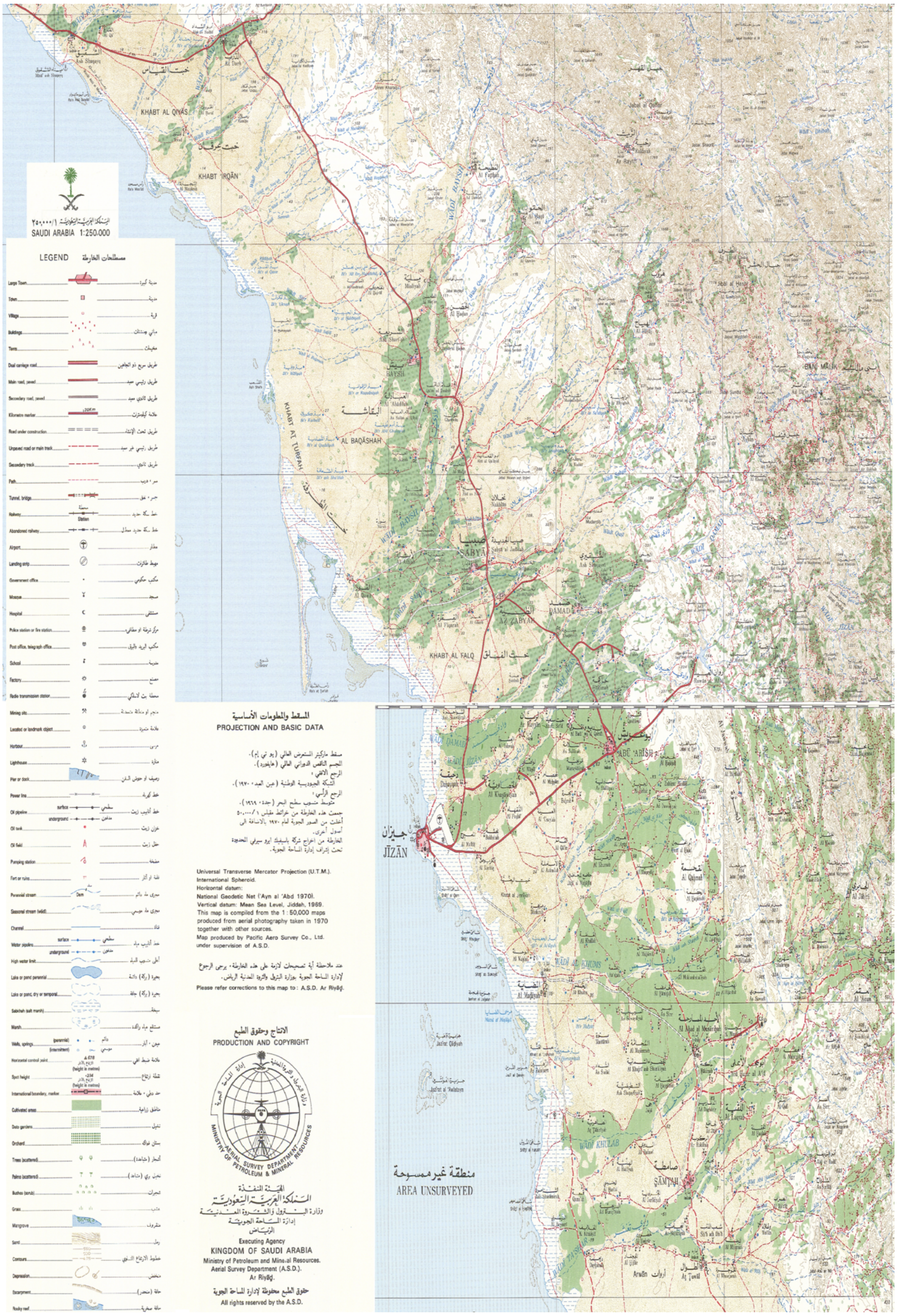


Figure 3.7. The Location of the Case Study. (Source: Saudi Geological Survey.)

Table 3.2. Data Information for 34 Cities (Source: GAS, 2018; Ministry of Municipal and Rural Affairs).

No.	Cities	Population	No.	Cities	Population
1	Jazan	127743	18	Al Reeth	14296
2	Al Fatiha	7021	19	Baish	37108
3	Sabya	165967	20	Al Haqu	10259
4	Alaliya	25744	21	Masliyah	17359
5	Qawz al Ja'afirah	18794	22	Alaydabi	7672
6	Al Kadami	17870	23	Addayer	34336
7	Abu Arish	144667	24	Ahad Al Masarihah	85965
8	Wadi Jizan	52445	25	Al Madaya/ Al-Hakamih	24745
9	Samtah	101033	26	Al Aridhah	45946
10	AlGofol	24817	27	Alhumira	9879
11	Al Sehi	21480	28	Al Shuqaiq	23875
12	Al Khubah/Alharth	8342	29	Al Tuwal	36547
13	Khushal	10244	30	Harub	15934
14	Damad	52193	31	Fayfa	29793
15	Al Shugayri	19408	32	Itwide	5081
16	Al Darb	36583	33	Aiban/Belghazi	24063

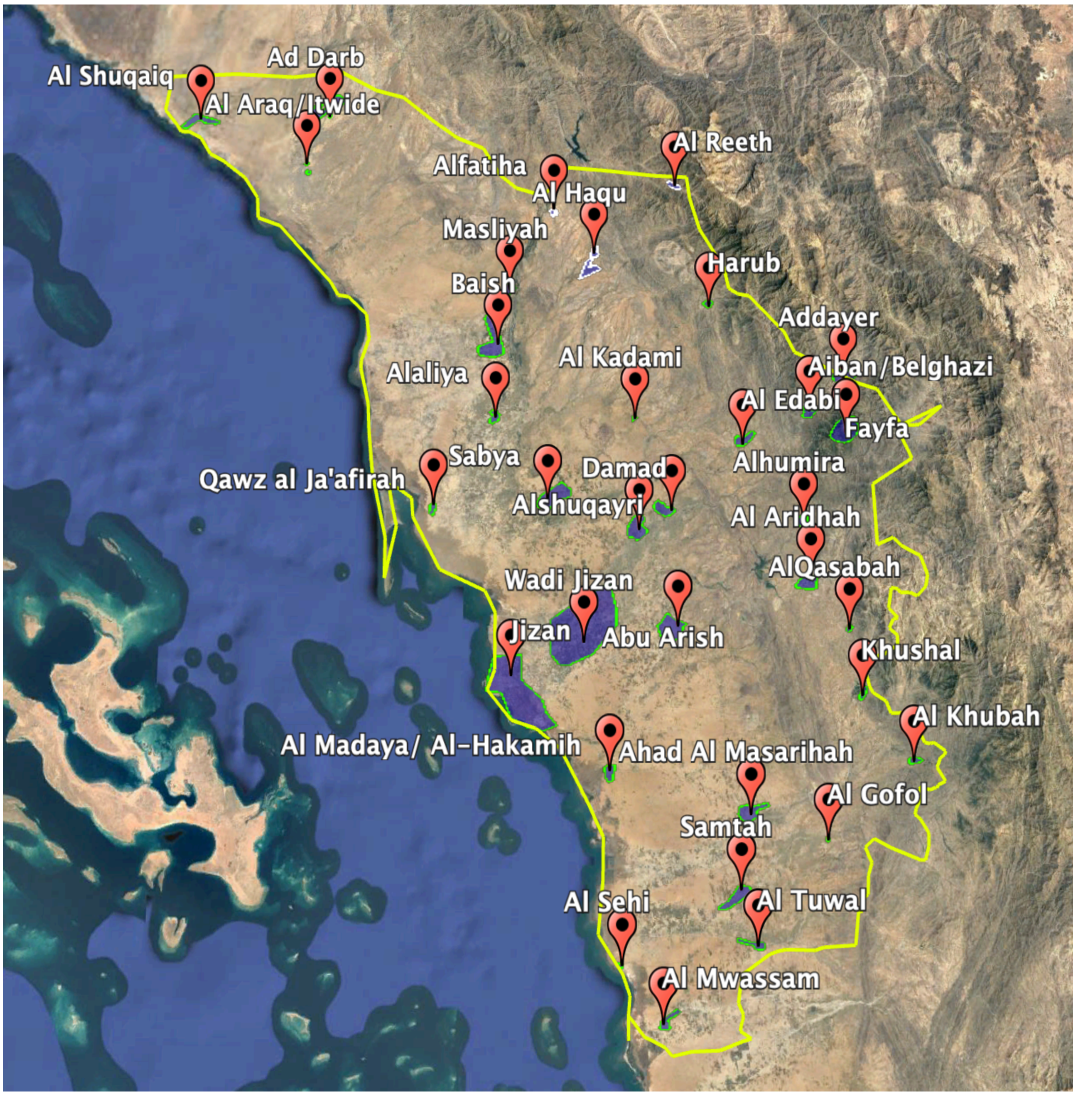


Figure 3.8. Locations of Cities in Jizan Region, KSA. (Source: Google Earth Pro.)

3.3.2 Topography Maps

Many sources can be used to identify the topography of regions, such as NASA, Google Earth, or geological websites. Topography maps identify elevations, locations of cities, activity areas, such as agriculture and industry, distances, roads, etc. The maps secured from the Saudi Geological Survey are already associated with TIFF files, which can be used in the ArcGIS program to identify many topographic components. Figure 3.7 combines two maps in order to cover the entire Jizan Region, at a scale of 1:250000. There are other topography maps for the region with a scale of 1:50000. For elevation purposes, topography maps have contour intervals within 10m in flat areas, allowing them to define elevations for each city considered in this study. Figure 3.9 shows the location of these cities using Google Maps.

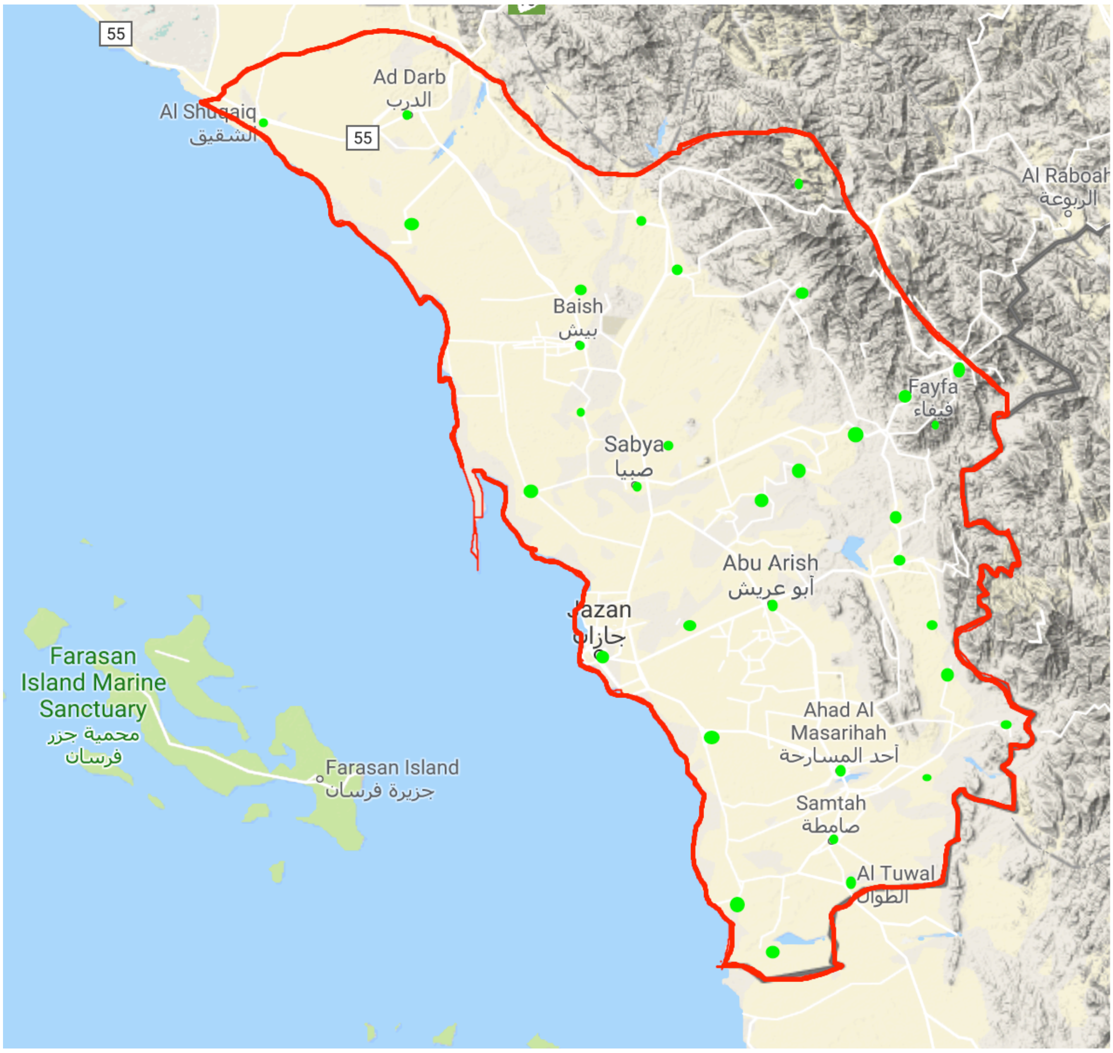


Figure 3.9. Terrain Map for the Jizan Region.

3.4 Sensitivity and Scenario Analysis

Scenario planning is a tool for improving the decision-making process and for dealing with uncertainty due to the application of different strategies (Melo & Varum, 2007). Scenarios are not meant to predict or accurately explain the future, but instead to assist decision-makers and stakeholders by reflecting uncertainties or assumptions (Varum & Melo, 2010). Scenario planning has been applied to many disciplines, including business, water resources, and urban and regional planning. Tapinos (2013) used a scenario planning approach in a cosmetic business unit in the UK. Aydin et al. (2014) used scenario analysis to calculate environmental and technical criteria for a water distribution system. The case study showed that the scenario planning process can assist stakeholders understand future uncertainties in their business units and improve their skills for developing strategies to deal with those uncertainties.

This study also utilizes sensitivity analysis, which is a technique used to determine how the independent variable impacts the dependent variable. The focus of the sensitivity analysis in this study is to 1) provide detailed results for guiding research and creating models and 2) calculate uncertainty for the prediction of each model. Policy decisions can represent both the efficiency of the predicted system and its predictive accuracy. Sensitivity analysis aims to describe how much model output values are influenced by modifications in model input values (Loucks & van Beek, 2017).

Sensitivity analyses can search for the optimal layout-design of sewer networks based on best location, type, and size for various input parameters. The total amount of design flow at manholes will depend on hydrological routing, which has an uncertain value. The storm sewer systems are analyzed with respect to changes in design flow, ground surface elevations, and lengths. The sensitivity approach was applied to the applications of M-3.

4 DEVELOPMENT OF OPTIMIZATION MODELS FOR LAYOUT AND WWTP LOCATIONS

4.1 Introduction

The problem of regional wastewater systems planning is finding the minimum costs of sewer layout and WWTPs, taking into consideration hydraulic constraints. The primary goal in this chapter is to develop an optimization model for the optimum configuration of regional wastewater systems, considering the minimum total cost flow that met connectivity model requirements and taking economic issues into account. The main focus in this chapter is to give a simple understanding of the approach of using the INL method for solving water collection/branched system problems by using the deterministic method. Additionally, this method can solve any system that has the same conception of the problem. The invention of the INL method was in the early 1970's, and it holds that a total number of INLs must be equal to the total number of pipes connected to outlines (Mays, 1976), described in detail in Chapter 5. There are various types of collection systems from the local level, as in storm sewer systems, to the regional level, as in wastewater systems. In addition, the method can be used for any system that has a dendritic or tree-type network. The values of wastewater produced and the capacity of WWTPs is known, as shown in hypothetical examples. The model is inspired by fundamental approach, the shortest path tree approach. Moreover, examples of application systems are provided in this phase and results are discussed for illustration. The models are formulated using MINLP in GAMS, solved by the BARON solver.

The plan of the chapter is as follows. First, it presents the problem addressed by the optimization model and writes up the equations of mathematical formulation to give a simple idea of using the INL approach. Second, it introduces the idea of collection type system (tree type) network and the usefulness of using the optimization model.

4.2 Cost Functions

The cost functions for wastewater systems, including installation, maintenance, and operating costs are usually strictly non-linear (Brand & Ostfeld, 2011). Mays et al. (1983) developed cost functions for regional water/wastewater systems, including installation, operating, and maintenance costs. These functions are strictly non-linear equations and are hard to define for different regions and economies of scale. This indicates that solutions would concentrate treatment into one or very few plants rather than in many plants. The influence of the degree of economies of scale can be seen in the results in Table 4.1 (Cunha, 2010).

Table 4.1 Wastewater System Costs for Different Cost Functions.

Number of WWTP	14	10	5	2	1
Low economies of scale $C = aQ^{0.95}$	100	101.4	104.4	112.1	121.1
Medium economies of scale $C = aQ^{0.86}$	101.1	100	100.8	106.8	112.9
High economies of scale$C = aQ^{0.75}$	106.9	103.1	100	104	107.3

The results concern a case study where all data is maintained except the cost function ($C = aQ^b$). Therefore, if only the level of the economy of scale is changed, the solutions will be different. As the economy of scale level increases (b value is lower), the solution is obtained for a smaller number of WWTPs. The solution of the wastewater problem at the regional level is a compromise. On the one hand, the solution where each community treats its own wastewater does not take into account the important economies of scale. On other hand, the solution where there is only one WWTP implies higher costs for taking wastewater from all discharge points to the WWTP (centralized system). Neither solution is an efficient, sustainable solution. Therefore, to find the best solution, there must be a trade-off between transportation costs and savings provided through economies of scales. Haghighi and Bakhshipour (2015), Karovic and Mays (2014), and Swamee and Sharma (2007) used cost functions to determine such things as considered costs per unit length, commercial diameter, or per unit volume, such as manholes. In reality, the system will be highly complex. It is anticipated that the parameters that will be the most dominant in determining the total cost will differ as a function of the particular systems' layout, components, cost functions, and imposed loadings (Brand & Ostfeld, 2011). Table 4.2 provides summary information about the overall cost functions associated with wastewater reuse system that can be used in the project.

Table 4.2. Overall Cost Functions Associated with Wastewater Reuse Systems.

Source	Overall cost
Mays et al. (1983)	= $2.88Q^{0.99}$ (capital cost of WWTP). = $0.0825Q^{0.96}$(operation and maintenance costs of WWTP). = $80Q^{0.461}$ (capital cost of pipeline). = $4.56x10^{-3}$ *distance(mi)*$Q^{0.495}$(operation and maintenance costs of pipeline). * All flow rates Q are in gallons per day
Al-A'ama and Nakhla (1995)	= 2.03$/$m^3$ The cost included capital cost (= 1.33 US$/$m^3$), tertiary treatment (= 0.16 US$/$m^3$), collection (= 0.3 US$/m^3) and distribution (= 0.06 US$/$m^3$).
Zahid (2007)	= 0.25$/$m^3-0.28/m^3 (capital costs of WWTP) = 0.03$/$m^3-0.05/m^3(operation and maintenance costs of WWTP).
Brand and Ostfeld (2011)	= 0.33$/$m^3$ (capital costs of WWTP)
Kajenthira et al. (2012)	Secondary TWW in the range of 0.13–0.63 US$/$m^3$, Tertiary TWW in the range of 1.19–2.03 US$/$m^3$.
Al-Zahrani et al. (2016)	TWW reuse ranges from 0.82 to 2.03 US$/$m^3$ with an average cost of 1.43 US$/$m^3$.

4.3 Connectivity Model

The application of MINLP to the optimal layout design of a sewer system includes two INLs, which represent ground surface elevations (i.e., from an upstream INL to the next downstream INL), a recursive procedure. Now, considering the flow of the system (i.e., from INL i to INL j), the computations are performed over the possible set of drops in crown elevations for each vector of possible connection of manholes on INLs i, j, and k. Flow directions for the set of nodes for all upstream and downstream node connections (outlets) denote which of these nodes' connections are possible for a sewer system layout. A vector of possible connections is needed for each connection. This vector has a dimension equal to the number of possible flow directions from each upstream node on INL i to downstream nodes on INL j and the same from the upstream node on INL j to downstream node on INL k. Each position in the vector of possible connections: which either has a 1, implying possible connection of the nodes, or a 0, implying no possible connection.

Figure 4.1 shows the drainage directions for a stage n between INLs i and j. For each of the upstream nodes (i_n= 1, 2, 3) on INL i, there are four flow directions: one to each downstream node ($j_{n},$. = 1,2,3,4). As an example, if the only possible connection of node i_n = 3 is to manhole j_n = 3 , then $T_{3,3}$= 1, $T_{3,1}$= 0, $T_{3,2}$= 0, and $T_{3,4}$= 0 . The concept of the vector of possible connectivity is shown in Figure 4.4.b. Indeed, more than one node on INL i may have a possible connection to a node on INL j, allowing for branches, so that the tree type network of a storm sewer system can be defined. Each node on INL i must have a possible connection to a node on INL j. The total vector of possible connectivity T_n at any stage n includes all possible connections.

The optimization computations are performed for each possible connection in stage n of INLs (i, j, and k) by considering crown elevations at nodes at the upstream and downstream of each possible open flow connection. Once this is completed, the minimum cost designs and costs associated with each downstream crown elevation for nodes on INL j for each possible connection are stored for use in the connectivity model.

Once the decisions for each possible connection at stage n of the system have been considered by the optimizer, the next step is to determine the minimum cost layout (connection of manholes) for that pipe connection. For connectivity optimization, it is difficult to incorporate the flow directions from upstream nodes as a second decision variable at the GAMS optimization. The main difficulty is the inability to compute the flow rates for the succeeding downstream pipes. To solve this difficulty, a special equation is built up in the optimization code, which is discussed in section 4.5.3. In order to compute these flow rates for the optimization in the next downstream node, connectivity must be defined for the previous upstream node. However, connectivity can be defined using MINLP in GAMS after the computational procedure over all pipes (minimum costs) is completed.

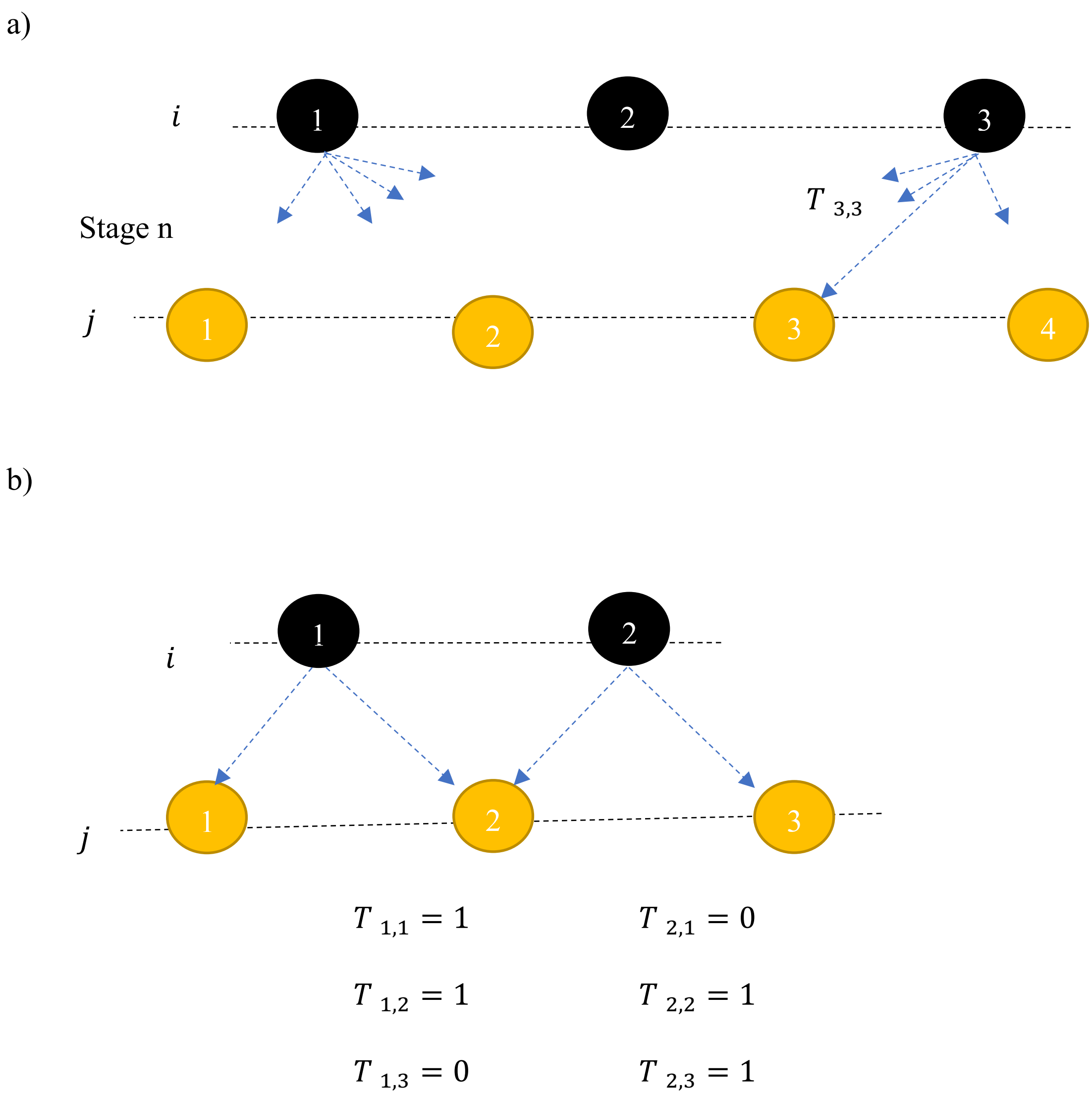


Figure 4.1 a) Drainage Directions, b) Possible Connections (adopted from Mays, 1976).

A connectivity model at manhole (i_n= 1 or 2) on INL i can be formulated using the costs required to continue draining each manhole (j_n= 1 or 2) on INL j through the next downstream manhole on INL j for each of the possible connections in manhole (i_n= 1 or 2). The possible connections for a simple network are shown in Figures 4.2 and 4.3. The total minimum cost for each connection to drain the nodes on INLs i and j to INL k through the nodes on INL j can be computed. From the optimization computations, the minimum cost design for each possible connection and each downstream node up to INL j are known. The minimum total cost up to INL k can be computed by performing the computations for each possible connection between INLs j and k, taking into account the minimum costs up to INL j.

This gives a minimum total cost for portions of the system up to INL k, considering each of the possible connections between INLs i and j in addition to the costs to continue draining the flow through the next downstream INL. Essentially, this amounts to performing designs for each possible drop within the corridors defined by the possible connections from INLs j to k. The cost of placing manholes on INL k is included. Once the minimum cost required to continue draining each manhole on INL j through the next downstream pipe for each possible connection in connection pipe is known, a model can be formulated to select the connectivity or layout for each pipe connection.

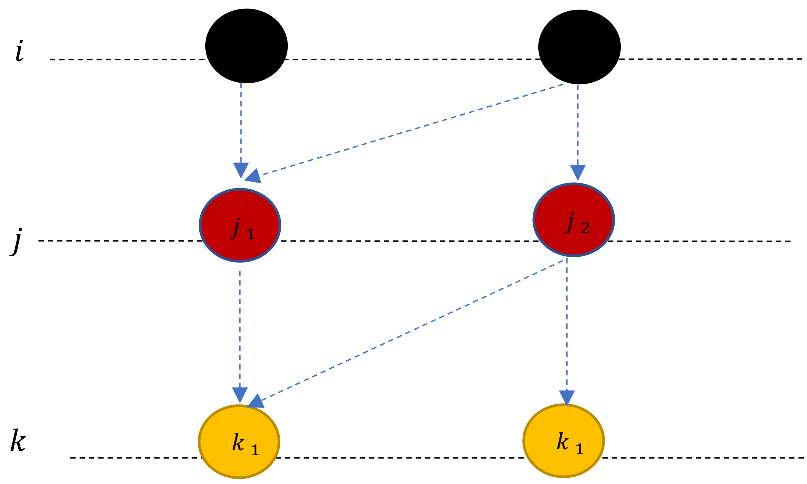


Figure 4.2. Possible Layouts to Drainage Line k

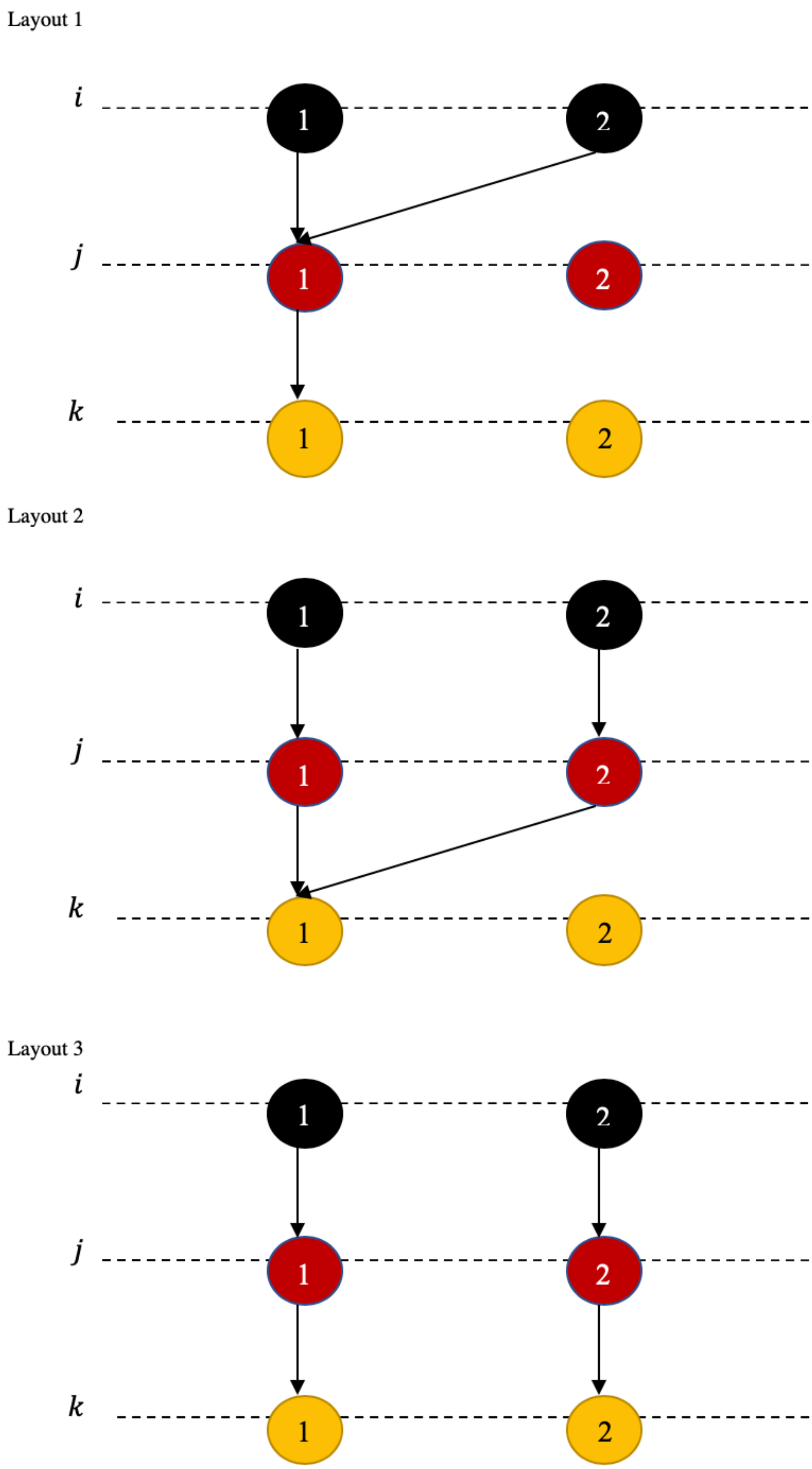


Figure 4.3. Possible Layouts.

4.3.1 Mathematical Formulation

The connectivity model is used to define the minimum cost connections of nodes once the computations have been completed for each possible connection between nodes n on INL i and nodes n on INL j before proceeding to the downstream nodes n on INL j to nodes n on INL k. The minimum cost layout must be chosen so that each upstream node on INL i is connected to downstream nodes on INL j by only one pipe, which must be one of the possible connections.

The conductivity model can be described as follows. These constraints allow for only one pipe to drain node n (i.e., the summation of the 0-1 variables, each representing a layout that allows node n to be drained, is equal to 1). Because each upstream node n must be drained, a constraint exists for each of these nodes n = 1, 2, 3, … N. Similarly, constraints can be developed to satisfy the restriction that each node on INL j is drained by only one pipe connecting to node n on INL k.

1- The flow from each source node i must flow through one collection node j, which can be satisfied as follows by using 0/1 binary variable $x_{i,j}$:

$$\sum_j a_{i,j} x_{i,j} = 1 \qquad \forall i \tag{4-1}$$

2- The flow from each collection node j must flow through one WWTP node on INL k, which can be satisfied as follows using 0/1 binary variable $y_{j,k}$:

$$\sum_k y_{j,k} b_{j,k} = \begin{cases} 1 & \text{If } \sum_k QI_{j,k} > 0 \\ 0 & \text{If } \sum_k QI_{j,k} = 0 \end{cases} \qquad \forall j \tag{4-2}$$

Where $a_{i,j}$ is equal to 1 if there is a pipe connecting from node i to node j, 0 otherwise and $b_{j,k}$ is equal to 1 if there is a pipe connecting from node j to node k, 0 otherwise.

Continuity constraints for flows in the system states that all the system must be in equilibrium, so that the flow produced at source nodes on INL i must be sent to WWTP nodes on INL k.

3- Continuity equation at source nodes on INLs i to collection nodes on INLs j.

$$QR_i=\sum_j QS_{i,j}x_{i,j}a_{i,j} \qquad \forall\, i \tag{4-3}$$

Continuity equation at collection nodes on INL j to WWTP nodes on INL k.

$$\sum_i QS_{ij}x_{i,j}a_{i,j} - \sum_k QI_{j,k}y_{j,k}b_{j,k} = 0 \qquad \forall\, j \tag{4-4}$$

4- Lower and upper bound constraint. However, bounds play a significant role in nonlinear models. To avoid an undefined operation, such as division by zero, it may be essential to provide bounds. In NLP, a definition of a reasonable solution space will assist in efficiently finding a solution (GAMS, 2017).

$$Qmin_{i,j}x_{i,j}a_{i,j} \leq QS_{i,j} \leq Qmax_{i,j}x_{i,j}a_{i,j} \qquad \forall (i,j) \tag{4-5}$$

$$Qmin_{j,k}y_{j,k}b_{j,k} \leq QI_{j,k} \leq Qmax_{j,k}y_{j,k}b_{j,k} \qquad \forall (i,j) \tag{4-6}$$

Qmin, Qmax: minimum and maximum amount of wastewater through the system

The objective is to select a set of possible layouts to satisfy the above constraints such that the minimum cost of the complete layout is selected for two stages of the system. A brief description of the method for determining the costs of possible layouts was given in the previous section and is discussed further in test examples. The cost of each possible layout is determined by selecting the cheapest layout of the possible connections associated with flows. The cost of all upstream pipes, the WWTP, and the cost of the possible layout represent the cost coefficients, CPIP, CPIP1, and CWWTP, for the objective function. The objective function can now be expressed as:

$$Min\ cost \sum_i \sum_j CPIP\ QS_{i,j} x_{i,j} a_{i,j} + \sum_j \sum_k CPIP2QI_{j,k} y_{j,k} b_{j,k}$$

$$+\sum_j \sum_k CWWTP\ QI_{j,k} b_{j,k} y_{j,k} \tag{4-7}$$

The connectivity model, expressed by the above equations, represents a 0-1 integer LP problem, which is developed and solved for three INLs of the system.

In this model formulation, it minimizes costs without considering the capacity limitation of a WWTP. The objective function minimizes total costs subject to continuity constraints and the connectivity model. At the starting point, the costs for each path are defined (paths from source nodes to collection nodes and paths from collection nodes to WWTP nodes), as are costs of new plant construction, operation, and maintenance. The reason for using this procedure is to check the quality of the model from a coding perspective. From a coding perspective, the mathematical formulation should be applied to the objective function, continuity constraints, and connectivity constraints only, so that later it can add more constraints for different purposes. Because it uses MINLP in GAMS, it should be upper- and lower-bounded for the variables. This was applied to make sure that the model would work perfectly without any issues and it examined the mathematical formulation for continuity constraints. Two different examples were used with different assumptions to make sure that the model would run in the right way and prove to be a reasonable solution.

4.3.2 Notation

Sets

i: Set of wastewater sources nodes on INL i.

j: Set of the possible location of intermediate (collection) nodes on INL j.

k: Set of possible location WWTP nodes on INL k.

Parameters

QR_i: Amount of wastewater produced at sources for a node on INL i.

$Q_{WWTP_{max}}$: Maximum amount of wastewater that may be treated at a node on INL k.

Q_{min}: Minimum flow allowed in the pipe system

Q_{max}: Maximum flow allowed in the pipe system

CPIP: The discount cost of installation, operation, and maintenance from source node i to intermediate node j.

CWWTP: The discount cost of new WWTP construction, operation, and maintenance.

CPIP2: The discount cost of the installation, operation, and maintenance from collection nodes on INL j to WWTP nodes on INL k.

$a_{i,j}$ = The possible paths of draining wastewater from source node to intermediate node j.

$$a_{i,j} = \begin{cases} 1 & \textit{possible path} \text{ from node i and node j} \\ 0 & \textit{no possible path} \text{ from node i and node j} \end{cases}$$

$b_{j,k}$: The possible paths of draining wastewater from collection nodes on INL j to WWTP nodes on INL k.

$$b_{j,k} = \begin{cases} 1 & \textit{possible path} \text{ from node j and node k} \\ 0 & \textit{no possible path} \text{ from node j and node k} \end{cases}$$

Variables

State variables:

$QS_{i,j}$: Flow carried from source node i to intermediate node j.

$QI_{j,k}$: Flow carried from intermediate node j to WWTP node k.

Decision variables:

$x_{i,j}$: The binary variable that will take value 1 if there is an existence of a particular pathway-linking node i to node j and 0 otherwise.

$y_{j,k}$: The binary variable that will take value 1 if there is an existence of a particular pathway linking node j to node k and 0 otherwise.

4.3.3 Test Example 1

To build the model in GAMS and ensure that the model formulation is correct, two examples were considered: the vector of possible connections with different costs associated with the flow and the vector of possible connections with same costs associated with flow (see Figure 4.4). Overall, continuity and connectivity constraints were used to ensure that the model would run perfectly through these two examples, changing the costs of pipelines and of WWTPs, and a possible path either way from source nodes on INL i to collection nodes on INL j or collection nodes on INL j to WWTP nodes on INL k). In test example 1, the possible paths for $a_{i,j}$, and, $b_{j,k}$, are considered, as shown in Tables 4.3 and 4.4 below:

Table 4.3. The Possible Paths of Draining Wastewater from Sources Nodes i to Intermediate Nodes j, $a_{i,j}$, for Test Example 1.

	n4	n5	n6
n1	1	No Possible Path	No Possible Path
n2	No Possible Path	1	No Possible Path
n3	No Possible Path	1	No Possible Path

Table 4.4. The Possible Paths of Draining Wastewater from Intermediate Nodes j to WWTP Nodes k, $b_{j,k}$, for Test Example 1.

	n7	n8	n9
n4	1	No Possible Path	1
n5	1	1	No Possible Path
n6	No Possible Path	No Possible Path	No Possible Path

It was assumed that all total costs of installation, operation, and maintenance from source nodes on INL i to collection nodes on INL j, CPIP, and total costs of installation, operation, and maintenance from collection nodes on INL j to WWTP nodes on INL k CPIP2, were included, as shown in Tables 4.5, 4.6, and 4.7.

Table 4.5. The Assumption of the Total Costs of Installing, Operating and Maintenance from Sources Nodes i to Intermediate Nodes j, CPIP, $/gallon for Test Example 1.

	n4	n5	n6
n1	$2	No Possible Path	No Possible Path
n2	No Possible Path	$5	No Possible Path
n3	No Possible Path	$1	No Possible Path

Table 4.6. The Assumption of the Total Costs of Installing, Operating and Maintenance from Intermediate Nodes j to WWTP Nodes k, CPIP2, $/gallon for Test Example 1.

	n7	n8	n9
n4	$3	No Possible Path	$3
n5	$3	$5	No Possible Path
n6	No Possible Path	No Possible Path	No Possible Path

Table 4.7. The Assumption of the Total Costs of New Plant Construction and Operating and Maintenance of Wastewater Treatment Plants, CWWTP, $/gallon for Test Example 1.

n7	$2
n8	$2
n9	$3

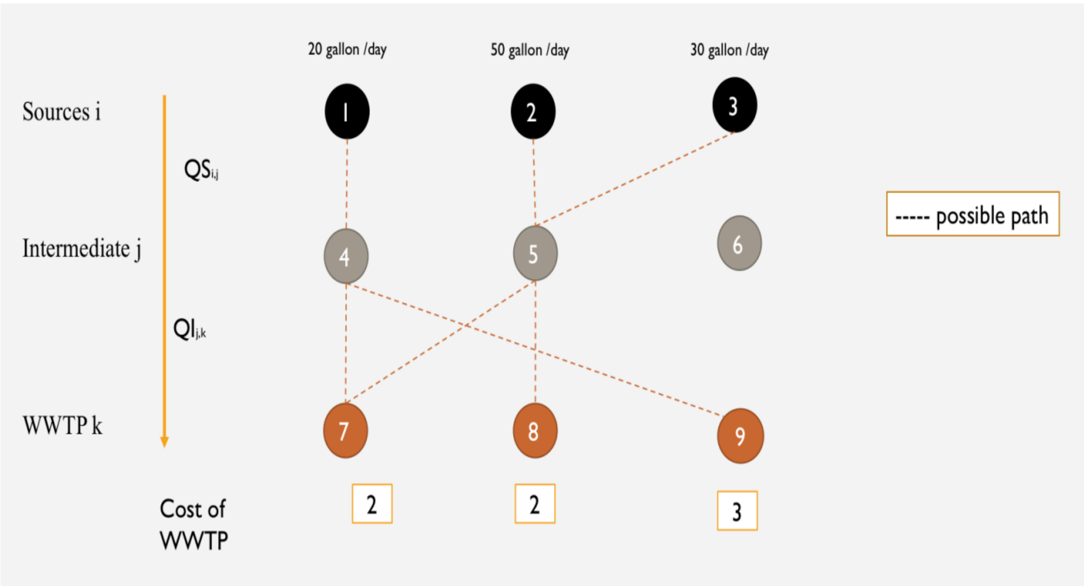


Figure 4.4. Input Values for the Model in GAMS for Test Example 1

The optimum configuration of scenario 1 shows that the flow tries to go through the possible paths allowed in the system and, at the same time, takes a minimum cost path, so all discharges are ended by n7, which has the lowest WWTP cost. See Figure 4.5.

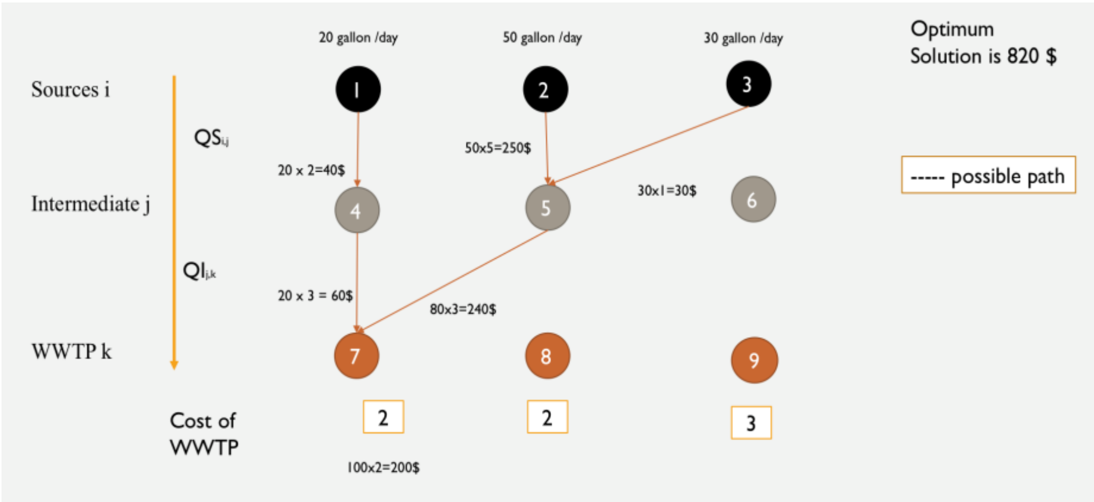


Figure 4.5. The Optimum Configuration for Test Example 1

4.3.4 Test Example 2

Example 2 performed the model for a different scenario that considered more possible paths and the same cost values for objective functions. The flowing tables and figures are our input data for the example system. In test example 2, the possible paths for $a_{i,j}$, and, $b_{j,k}$, are as follows:

- n1 to n6
- n2 to n6
- n2 to n4
- n3 to n4
- n4 to n9
- n5 to n9
- n5 to n7
- n6 to n7

Table 4.8. Possible Paths of Draining Wastewater from Sources Node i to Intermediate Nodes j, $a_{i,j}$, for Test Example 2.

	n4	n5	n6
n1	1	1	No Possible Path
n2	No Possible Path	1	No Possible Path
n3	No Possible Path	1	1

Table 4.9. Possible Paths of Draining Wastewater from Intermediate Node j to WWTP Nodes k, $b_{j,k}$, for Test Example 2.

	n7	n8	n9
n4	1	1	No Possible Path
n5	No Possible Path	1	No Possible Path
n6	No Possible Path	1	1

The total costs of installation, operation, and maintenance from source node i to intermediate node j, CPIP, are assumed to be equal to $1 /gallon.

Table 4.10. Assumption of Total Costs of Installation, Operation, and Maintenance from Source Node i to Intermediate Node j, CPIP, $/gallon for Test Example 2.

	n4	n5	n6
n1	$1	$1	No Possible Path
n2	No Possible Path	$1	No Possible Path
n3	No Possible Path	$1	$1

Total costs of installation, operation, and maintenance from intermediate node j to WWTP node k, CPIP2, are as follows.

Table 4.11. Assumption of Total Costs of Installation, Operation, and Maintenance from Intermediate Node j to WWTP Node k, CPIP2, $/gallon for Test Example 2.

	n7	n8	n9
n4	$1	$1	No Possible Path
n5	No Possible Path	$1	No Possible Path
n6	No Possible Path	$1	$1

Assumed costs of new WWTP construction, operation, and maintenance would be equal to $1/gallon.

Table 4.12. Assumption of Total Costs of New Wastewater Treatment Plant Construction, Operation, and Maintenance, CWWTP, $/gallon for Test Example 2.

n7	$1
n8	$1
n9	$1

All pipelines and WWTP costs are the same. The optimum configuration for test example 2 is shown in Figure 4.7, with total costs of $300.

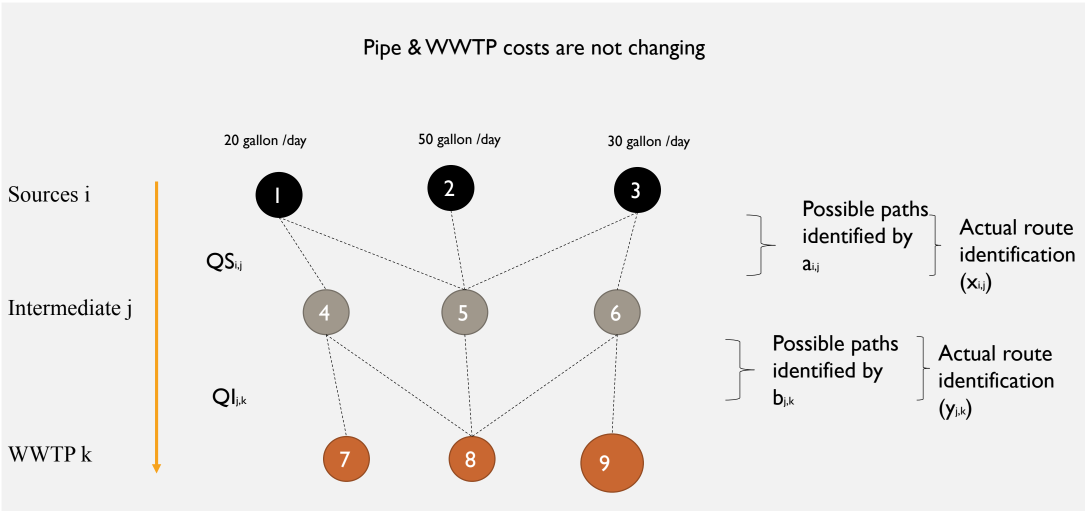


Figure 4.6. Input Values for the Model in GAMS for Test Example 2.

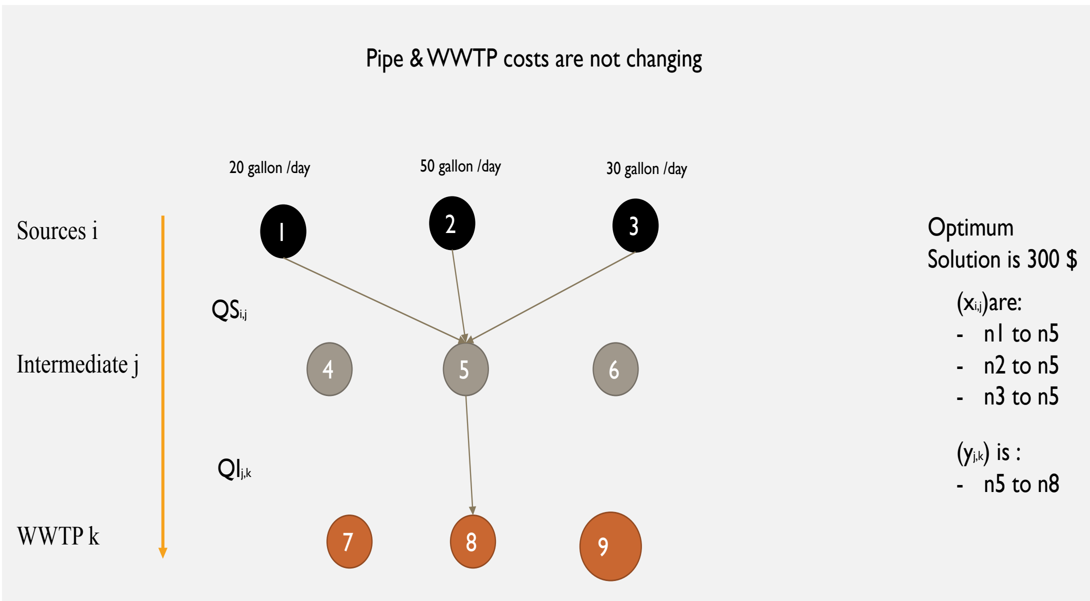


Figure 4.7. The Optimum Configuration for Test Example 2.

4.4 Optimization M-1

The first phase of this dissertation implemented the continuity and capacity limitations of the WWTP constraints. This phase was developed successfully through the GAMS program using MINLP. The model here is described to ensure that the wastewater generated from sources is treated in WWTPs. Figure 4.8 shows the layout of the model formulation. The setup of the system includes three source nodes on INL i, three collection nodes on INL j, and three nodes on INL k. The flow, $QS_{i,j}$, from source nodes on INLs i to collections on INLs j and the flow, $QC_{j,k}$, from collection nodes on INLs j to the WWTP on INLs k, the flow, QT_k, is treated water from WWTP. The mathematical formulation is similar to the connectivity model, but three more constraints have been added (1.3, 1.4, and 1.5). Under this approach, vector possible connections, $a_{i,j}$, and, $b_{j,k}$, were not considered.

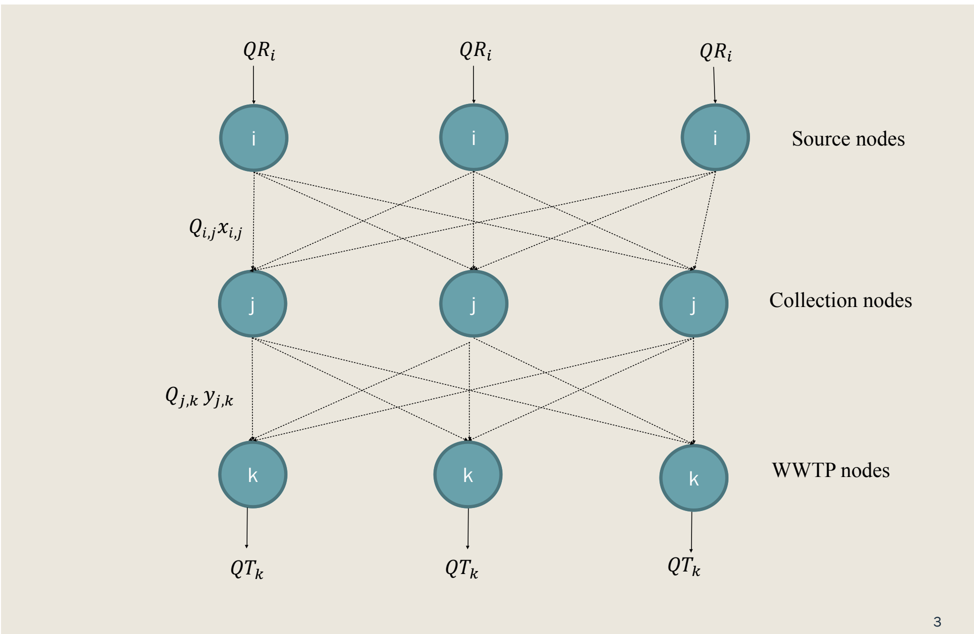


Figure 4.8. Layout of Model Formulation.

4.4.1 Mathematical Formulation

Objective function

The objective function minimizes the total costs associated with the installation and construction of a sewer system and WWTP by determining an optimal layout of sewer pipes' network and the locations of the candidate WWTP/s.

$$\text{Min Cost} = \sum_i \sum_j C_{Sewer(I,j)}\, QS_{I,j}\, x_{I,j} + \sum_j \sum_k C_{Sewer(j,k)} QC_{j,k}\, y_{j,k} + \sum_k C_{WWTP,k}(QT_k) \tag{4-8}$$

Where

$C_{Sewer(i,j)}$ is the discounted cost of the installation, operation, and maintenance of sewer system from source node i to collection node j.

$QS_{i,j}$ is the flow rate from source node i to collection node j.

$x_{i,j}$ is the connectivity of source node i to collection node j.

$C_{Sewer(j,k)}$ is the discounted cost of the installation, operation, and maintenance of the sewer system from collection node j to WWTP node k.

$QC_{j,k}$ is the flow rate from collection node j to WWTP node k.

$y_{j,k}$ is the connectivity of collection node j to WWTP node j.

$C_{WWTP,k}$ is the discounted cost of treated wastewater at WWTP node k.

QT_k is the flow rate of treated wastewater at WWTP node k.

Subject to:

Continuity constraint

1. The produce flow, QR_i, at source node i should be equal to the sum of the conveyed flow, $QS_{i,j}$, from source node i to collection node j. A continuity equation was written for each source node i.

$QR_i = \sum_j QS_{i,j} x_{i,j}$ $\forall$ i (4-9)

2. The difference between the sum of the total collected inflow, $QS_{i,j}$, at collection node j minus the sum of the total outflow to wastewater treatment plan k has to be equal to zero. The continuity equation for each collection node j is written as

$\sum_i QS_{i,j} x_{i,j} - \sum_k QC_{j,k} y_{j,k} = 0$ $\forall$ j (4-10)

3. The sum of the conveyed flow, $QC_{i,j}$, from collection node j to WWTP node k, have to be equal to outflow (treated wastewater) at each WWTP node. The continuity equation for each WWTP node is written as

$\sum_J QC_{j,k} y_{j,k} = QT_k$ $\forall$ k (4-11)

4. The sum of the total produced wastewater, QR_i, at source nodes i=1,2,….,I, should be equal to the sum of wastewater treated, QT_k, at wastewater treatment nodes k=1,2,…, K, as follows: the wastewater produced at source nodes should be treated.

$\sum_i QR_i = \sum_k QT_k$ (4-12)

5. The sum of the conveyed flow, $QC_{i,j}$, from collection node j to WWTP node k, has to be equal or less than the maximum WWTP capacity, MaxQT,. The capacity equation for this constraint is written as follows:

$\sum_j QC_{j,k} y_{j,k} \leq MaxQ_{wwtp}$ $\forall$ k (4-13)

Connectivity model

6. The flow from each source node i must flow through one collection node j, which can be satisfied as follows, using 0/1 binary variable, $x_{i,j}$,:

$\sum_j x_{i,j} = 1$ $\forall$ i (4-14)

7. The flow from each collection node j must flow through one WWTP node k, which can be satisfied as follows using 0/1 binary variable, $x_{i,j,}$:

$$\sum_k y_{j,k} = \begin{cases} 1 & \text{If } \sum_k QC_{j,k} > 0 \\ 0 & \text{If } \sum_k QC_{j,k} = 0 \end{cases} \qquad \forall\, j \qquad (4\text{-}15)$$

- The flow through the sewer system should be between maximum and minimum flows, which can be satisfied as follows using Qmax and Qmin.

$$Qmin_{i,j}x_{i,j} \leq QS_{i,j} \leq Qmax_{i,j}x_{i,j} \qquad \forall\ (i, j) \qquad (4\text{-}16)$$

$$Qmin_{j,k}y_{j,k} \leq QC_{j,k} \leq Qmax_{j,k}y_{j,k} \qquad \forall\ (j, k) \qquad (4\text{-}17)$$

4.4.2 Write up Mathematical Equations for Example System

The mathematical equations for example Figure 4.9 can be described as following:

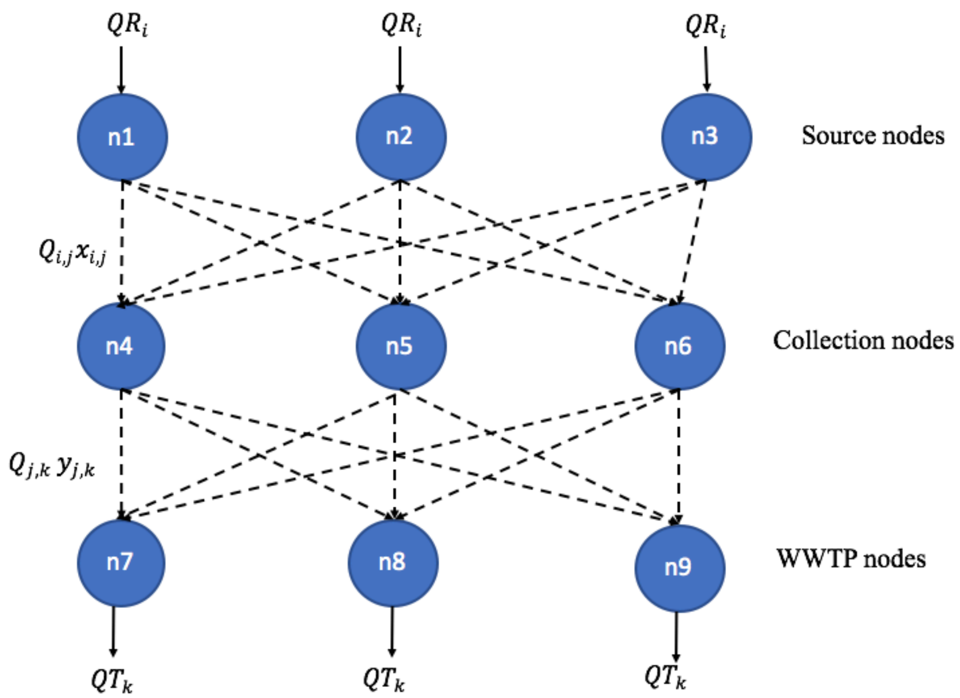


Figure 4.9. Example System Layout.

Objective function:

The objective function is to minimize total costs:

$\mathrm{Cost}=C_{n1,n4}QS_{n1,n4}x_{n1,n4}+C_{n1,n5}QS_{n1,n5}x_{n1,n5}+C_{n1,n6}QS_{n1,n6}x_{n1,n6}+$

$C_{n2,n4}QS_{n2,n4}x_{n2,n4}+$

$C_{n2,n5}QS_{n2,n5}x_{n2,n5}+C_{n2,n6}QS_{n2,n6}x_{n2,n6}+C_{n3,n4}QS_{n3,n4}x_{n3,n4}+$

$C_{n3,n5}QS_{n3,n5}x_{n3,n5}+C_{n3,n6}QS_{n3,n6}x_{n3,n6}+C_{n4,n7}QC_{n4,n7}y_{n4,n7}+C_{n4,n8}QC_{n4,n8}y_{n4,n8}+$

$C_{n4,n9}QC_{n4,n9}y_{n4,n9}+C_{n5,n7}QC_{n5,n7}y_{n5,n7}+C_{n5,n8}QC_{n5,n8}y_{n5,n8}+$

$C_{n5,n9}QC_{n5,n9}y_{n5,n9}+C_{n6,n7}QC_{n6,n7}y_{n6,n7}+C_{n6,n8}QC_{n6,n8}y_{n6,n8}+$

$C_{n6,n9}QC_{n6,n9}y_{n6,n9}+C_{WWTP,n7}\,QT_{n7}+C_{WWTP,n8}\,QT_{n8}+C_{WWTP,n9}\,QT_{n9}$

Constraints:

Continuity EQ at source node i

Source at node 1 (n1) $QS_{n1,n4}x_{n1,n4}+QS_{n1,n5}x_{n1,n5}+QS_{n1,n6}x_{n1,n6}=QR_{n1}$

Source at node 2 (n2) $QS_{n2,n4}x_{n2,n4}+QS_{n2,n5}x_{n2,n5}+QS_{n2,n6}x_{n2,n6}=QR_{n2}$

Source at node 3 (n3) $QS_{n3,n4}x_{n3,n4}+QS_{n3,n5}x_{n3,n5}+QS_{n3,n6}x_{n3,n6}=QR_{n3}$

Continuity EQ at collection node j

Collection at node 4 (n4) $QS_{n1,n4}x_{n1,n4}+QS_{n2,n4}x_{n2,n4}+QS_{n3,n4}x_{n3,n4}-$

$QC_{n4,n7}y_{n4,n7}-QC_{n4,n8}y_{n4,n8}-QC_{n4,n9}y_{n4,n9}=0$

Collection at node 5 (n5) $QS_{n1,n5}x_{n1,n5}+QS_{n2,n5}x_{n2,n5}+QS_{n3,n5}x_{n3,n5}-$

$QC_{n5,n7}y_{n4,n7}-QC_{n5,n8}y_{n5,n8}-QC_{n5,n9}y_{n5,n9}=0$

Collection at node 6 (n6) $QS_{n1,n6}x_{n1,n6}+QS_{n2,n6}x_{n2,n6}+QS_{n3,n6}x_{n3,n6}-$

$QC_{n6,n7}y_{n4,n7}-QC_{n6,n8}y_{n6,n8}-QC_{n6,n9}y_{n6,n9}=0$

Continuity EQ at WWTP node k

WWTP at node 7 (n7) $QC_{n4,n7}y_{n4,n7}+QC_{n5,n7}y_{n5,n7}+QC_{n6,n7}y_{n6,n7}=QT_{n7}$

WWTP at node 8 (n8) $QC_{n4,n8}y_{n4,n8}+QC_{n5,n8}y_{n5,n8}+QC_{n6,n8}y_{n6,n8}=QT_{n8}$

WWTP at node 9 (n9) $QC_{n4,n9}y_{n4,n9} + QC_{n5,n9}y_{n5,n9} + QC_{n6,n9}y_{n6,n9} = \mathrm{QT_{n9}}$

- Ensure that all the wastewater produced in the region will be sent to a treatment plant.

$$\mathrm{QR_{n1}} + \mathrm{QR_{n2}} + \mathrm{QR_{n3}} = \mathrm{QT_{n7}} + \mathrm{QT_{n8}} + \mathrm{QT_{n9}}$$

Constraint for sewer layout

Each source node must flow to one collection node

Conductivity(n1) $x_{n1,n4}+x_{n1,n5}+x_{n1,n6} = 1$

Conductivity(n2) $x_{n2,n4}+x_{n2,n5}+x_{n2,n6} = 1$

Conductivity(n3) $x_{n3,n4}+x_{n3,n5}+x_{n3,n6} = 1$

Each collection node must flow to one WWTP node

Conductivity(n4) $y_{n4,n7}+y_{n4,n8}+y_{n4,n9} = 1$ if $Q_{n4,n7}+Q_{n4,n8}+Q_{n4,n9} > 0$

Conductivity(n4) $y_{n4,n7}+y_{n4,n8}+y_{n4,n9} = 0$ if $Q_{n4,n7}+Q_{n4,n8}+Q_{n4,n9} = 0$

Conductivity(n5) $y_{n5,n7}+y_{n5,n8}+y_{n5,n9} = 1$ if $Q_{n5,n7}+Q_{n5,n8}+Q_{n5,n9} > 0$

Conductivity(n5) $y_{n5,n7}+y_{n5,n8}+y_{n5,n9} = 0$ if $Q_{n5,n7}+Q_{n5,n8}+Q_{n5,n9} = 0$

Conductivity(n6) $y_{n6,n7}+y_{n6,n8}+y_{n6,n9} = 1$ if $Q_{n6,n7}+Q_{n6,n8}+Q_{n6,n9} > 0$

Conductivity(n6) $y_{n6,n7}+y_{n6,n8}+y_{n6,n9} = 0$ if $Q_{n6,n7}+Q_{n6,n8}+Q_{n6,n9} = 0$

Constraint for WWTP capacity

The treated wastewater should not exceed the maximum capacity of WWTP

WWTP at node 7 (n7) $QC_{n4,n7}y_{n4,n7} + QC_{n5,n7}y_{n5,n7} + QC_{n6,n7}y_{n6,n7} \leq$ MaxQ_{wwtp}

WWTP at node 8 (n8) $QC_{n4,n8}y_{n4,n8} + QC_{n5,n8}y_{n5,n8} + QC_{n6,n8}y_{n6,n8} \leq$ MaxQ_{wwtp}

WWTP at node 9 (n9) $QC_{n4,n9}y_{n4,n9} + QC_{n5,n9}y_{n5,n9} + QC_{n6,n9}y_{n6,n9} \leq$

MaxQ_{wwtp}

4.4.3 Special Constructions for Conditional Variables in GAMS

To define the statement in GAMS, an equation must be constructed that can handle the if statement. The GAMS optimization model does not allow an if statement for variables, so it needs to find an equation that can control the constraint. This method was found in GAMS tutorials for beginners (Savitsky & McKinney, 1999, p. 37). The simple function can be used in the deterministic model to solve this problem, which is described below.

$$\sum_{\mathrm{k}} \mathrm{y}_{\mathrm{j,k}} = \begin{cases} 1 & \text{If } \sum_{k} \mathrm{QC}_{\mathrm{j,k}} > 0 \\ 0 & \text{If } \sum_{k} \mathrm{QC}_{\mathrm{j,k}} = 0 \end{cases} \qquad \forall\, \mathrm{j} \tag{4-18}$$

The function f(x) equals 1 for any value of x that is more than 1, and equals 0 for any value of x that equals 0.

$$f(x) = \begin{cases} 1 & x > 1 \\ 0 & x = 0 \end{cases}$$

Where,

$$f(x) = \frac{x}{|x-1|+1}$$

The method does not work for $0 < x < 1$ and $x < 0$. However, there is another method called an indicator function, which is an event of the random variable value 1 if the event occurs and value 0 if the event does not occur. Indicator functions are often used to simplify notation and demonstrate theorems in probability theory (statlect.com).

$$\sum_{\mathrm{k}} \mathrm{y}_{\mathrm{j,k}} = 1 * \frac{\sum_{\mathrm{k}} \mathrm{Q}_{\mathrm{j,k}}}{|\sum_{\mathrm{k}} \mathrm{Q}_{\mathrm{j,k}} - 1| + 1} \qquad \forall\, \mathrm{j} \tag{4-19}$$

4.5 Example Application

It is assumed that there are three source nodes n1, n2, and n3, which produce wastewater in the amounts of 18 Mgal/d, 46 Mgal/d, and 74 Mgal/d, respectively. The average populations in these source nodes are 700,000 people for n1, 2,000,000 people for n2, and 3,000,000 people for n2. See Figure 4.10. The cheapest costs for the possible connections are represented by red dash lines, which are \$1.2/gallon. The costs of treated wastewater are \$3.2/gallon. As shown in Figure 4.11, this system has an objective value of \$745.2 million. This example was solved in GAMS using MINLP using the BARON solver. The model seems to work well, as it is takes the minimum costs paths. Figure 4.12 shows a screenshot of the GAMS output that shows binary variable = 1 only (from node 6 to node 7), which makes sense because the only value of costs that is greater than one is from collection node 6 to WWTP node 7, and there is no flow to the other nodes. The infeasible solution would occur with different solvers. The layout represents the optimum solution for this application, while some solvers do not give the same solution. Table 4.1. shows the solvers that provide feasible and infeasible solutions for application example1.

Table 4.13. Solvers Used in Example Application 1.

Solver Name	Infeasible solution	Feasible solution	If feasible, same solution?	Time consumed
SBB	√			Seconds
OQNLP	√			Seconds
LOCALSOLVER		√	No	16 mins
SCIP		√	No	Seconds
LINDOGLOBAL		√	Yes	Seconds
LINDO		√	Yes	Seconds
KNITOR		√	Yes	Seconds
COUENNE		√	Yes	Seconds
ANTIGONE		√	Yes	Seconds
ALPHAECP		√	No	Seconds
BONMIN		√	No	Seconds
BONMINH		√	No	Seconds
BARON		√	Yes	Seconds
DICOPT/CONOPT1	√			Seconds

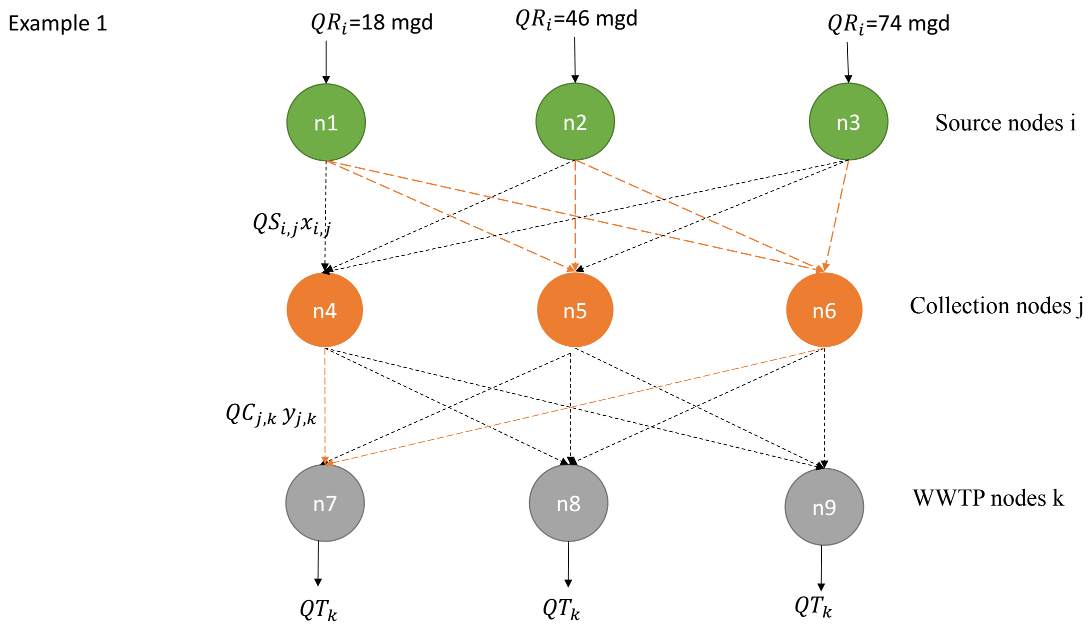


Figure 4.10. Input Data for Example Application.

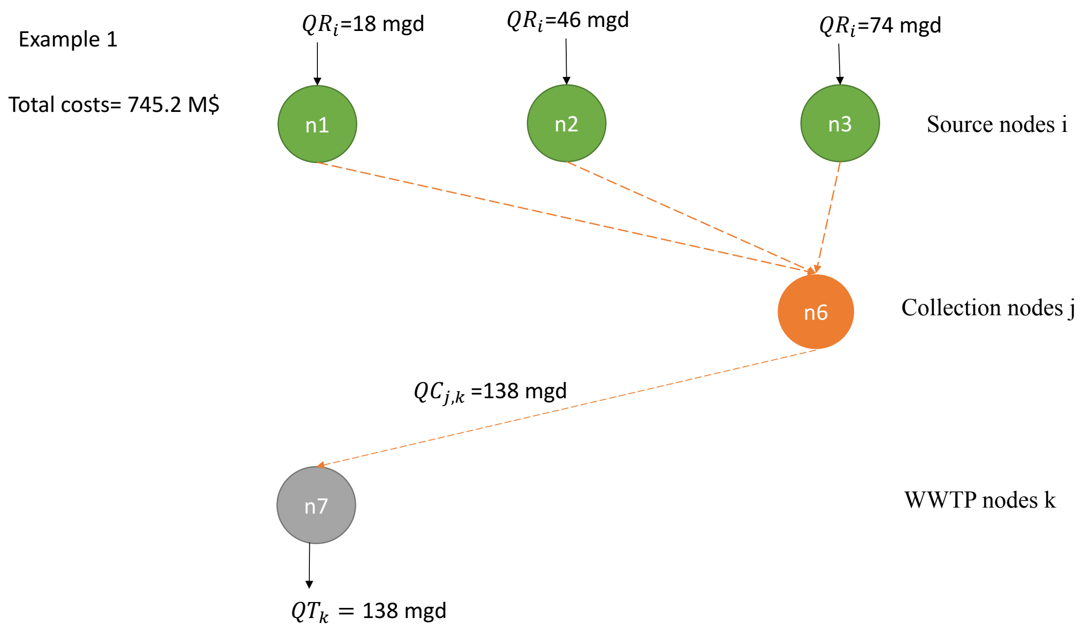


Figure 4.11. Results of Example Application.

```
            n7

n6      138.000


----     119 VARIABLE QT.L  wastewater treated in WWTP ( k )

n7 138.000


----     119 VARIABLE COST.L                =      745.200  The Optimum cost

----     119 VARIABLE x.L

            n6

n1        1.000
n2        1.000
n3        1.000


----     119 VARIABLE y.L

            n7

n6        1.000
```

Figure 4.12. Screenshot of GAMS Output

5 ADOPTION OF AN OPTIMIZATION MODEL FOR A STORM WATER SYSTEM

5.1 Introduction

The problem of storm water systems is finding the minimum costs for sewer layout considering hydraulic constraints. The primary goal of this chapter was to find the optimum configuration of a storm water system considering the minimum total cost flow that met conductivity requirements using the INL method. The third phase was to provide an optimization model that minimizes costs using the INL method for a collection type system, which is an expansion idea of the first phase. From a mathematical point of view, the third phase developed the INL method for a storm water system. Moreover, an example application system is provided in this phase and the results are discussed. The models were developed by using an MINLP approach.

A sewer system can be considered to be many nodes joined together by a number of links. The nodes are the manholes, junctions, and network outlets, and the links are the sewer pipes (Mays & Yen, 1975). The main idea of this phase was to give a simple approach to using the INL method to solve storm water system problems. In addition, it can solve any system that has a similar concept.

5.2 Iso-nodal Line (INL) Method for Layout of a Storm water System

Mays (1976) used the INL method, which can be applied to any water collection system. An INL is defined as an imaginary line connecting manholes that have the same number of pipes connecting to the outlet of the drainage system. The same concept can be applied to a storm water system, which can be defined as imaginary lines connecting nodes that have same number of pipes connecting to one or more flows drainage nodes. In addition, the INL method can work for any system that has flows from upstream (I) nodes to downstream (I + 1) nodes, with no flow between two nodes in the same INL. As described, a sewer system is a collection system that we can use to introduce INLs. The main aim of this methodology is to determine a layout for a sewer system and locations for drainage outlets at minimum cost. It contains continuity and connectivity equations only, considered by decision variables $x_{(I1,I2)}, y_{(I2,I3)},$ and $z_{(I3,I4)}$, which have a value of (0 or 1). A value of 1 means that a possible connection is selected in an optimum solution, otherwise it has a value of 0. For costs, discount costs are assumed to be per unit discharges for each possible connection: $C_{Sewer(N_{I1},N_{I2})}, C_{Sewer(N_{I2},N_{I3})},$ and $C_{Sewer(N_{I3},N_{I4})},$, that is, the possible connection at nodes on the upstream INL $n_I = 1 \ldots\ldots N_I$ to nodes on the downstream INL $n_{I+1} = 1 \ldots\ldots N_{I+1}$. Discount costs for drainage flows, $C_{drag,N_{I+1}}$, are defined as input values. The continuity equation is required to be written between two INLs. In other words, for the first INL, there is a known inflow (design flows by source), $Q_{N_{I1}}$, and outflow (flow carried to the second INL), $QS_{N_{I1},N_{I2}}, x_{N_{I1},N_{I2}},$. For the second INL, there is an inflow (flow carried from the first INL), $QS_{N_{I1},N_{I2}}, x_{N_{I1},N_{I2}}$, and outflow (flow carried to the third INL), $QC_{N_{I2},N_{I3}}, y_{N_{I2},N_{I3}},$. The third INL has inflow (flow carried from the second INL), $QC_{N_{I2},N_{I3}}, y_{N_{I2},N_{I3}}$, and outflow (flow carried to the fourth INL), $QO_{N_{I3},N_{I4}}, z_{N_{I3},N_{I4}}$. At the fourth INL, the total inflow (flow carried from the third INL), $QO_{N_{I3},N_{I4}} z_{N_{I3},N_{I4}}$, should be equal to drainage flows Qd_{I4}. Additionally, there are conductivity constraints between each INL. For example, at INL number 3, each node must flow to one node that is located in INL number 4, Please see Figure 5.2. In addition, considering upper and lower bounds for these variables is important since the problem is formulated as an MINLP. However, the last brief description is only for a system with four INLs, so there are more INLs, they could be added to the model using same concept.

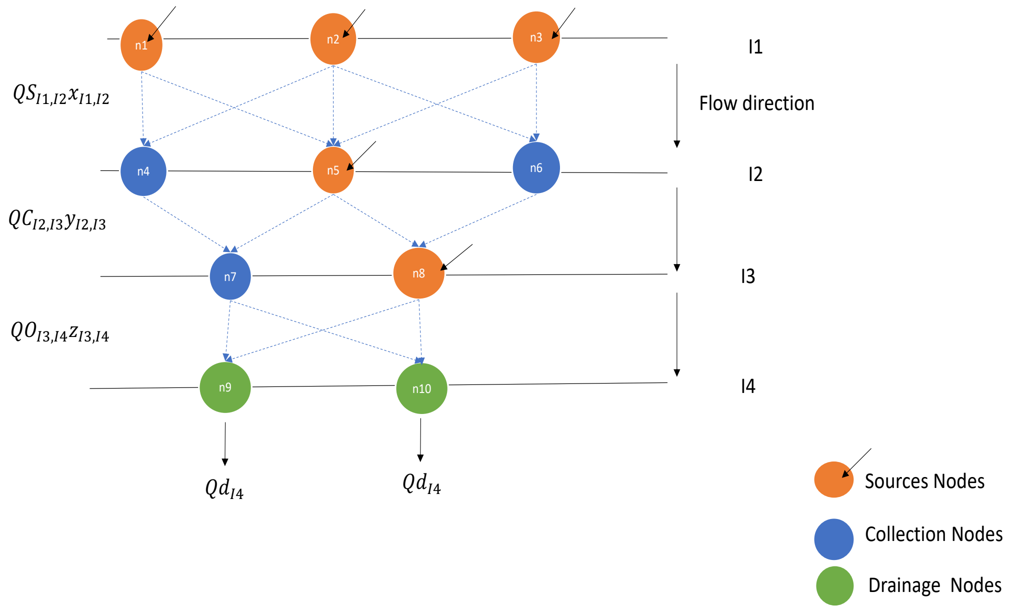


Figure 5.1. Layout of Collection System Using Iso-nodal Line Method.

5.2.1 Mathematical Formulation for Example 1

5.2.1.1 Objective Function

The objective function of this model is to minimize the total costs associated with the construction of a sewer system and drainage area by determining an optimal layout of the sewer pipes network and the location(s) of candidate drainage node(s).

$$\text{Min Cost} = \sum_{N_{I1}} \sum_{N_{I2}} C_{Sewer(N_{I1},N_{I2})}\, QS_{N_{I1},N_{I2}} x_{N_{I1},N_{I2}} + \sum_{N_{I2}} \sum_{N_{I3}} C_{Sewer(N_{I2},N_{I3})}\, QC_{N_{I2},N_{I3}} y_{N_{I2},N_{I3}} + \sum_{N_{I3}} \sum_{N_{I4}} C_{Sewer(N_{I3},N_{I4})}\, QO_{N_{I3},N_{I4}} z_{N_{I3},N_{I4}} + \sum_{N_{I4}} C_{drag,N_{I4}} (Qd_{I4}) \tag{5-1}$$

Where:

$C_{Sewer(I1,I2)}$ is the discounted cost associated with the sewer system from node N on INL (I1) to node N on INL (I2).

$C_{Sewer(I2,I3)}$ is the discounted cost associated with sewer system from node N on INL (I2) to node N on INL (I3).

$C_{Sewer(I3,I4)}$ is the discounted cost associated with sewer system from node N on INL (I3) to node N on INL (I4).

$C_{drag,N_{I4}}$ is the discounted cost associated with drainage flows at node N on INL (I4).

$QS_{N_{I1},N_{I2}}$ is the flow rate from node N on INL (I1) to node N on INL (I2).

$QC_{N_{I2},N_{I3}}$ is the flow rate from node N on INL (I2) to node N on INL (I3).

$QO_{N_{I3},N_{I4}}$ is the flow rate from node N on INL (I3) to node N on INL (I4).

$QT_{N_{I5}}$ is the drainage flow at node N on INL (I5).

$x_{N_{I1},N_{I2}}$ is a binary variable that will take value 1 if there is existence of a particular pathway that links node N on INL (I1) to node N on INL (I2).

$y_{N_{I2},N_{I3}}$ is a binary variable that will take value 1 if there is existence of a particular pathway that links node N on INL (I2) to node N on INL (I3).

$z_{N_{I3},N_{I4}}$ is a binary variable that will take value 1 if there is existence of a particular pathway that links node N on INL (I3) to node N on INL (I4).

5.2.1.2 Constraints

Continuity Constraint

8. The produce flow at node N on INL I1, $\mathrm{Q}_{\mathrm{N}_{\mathrm{I1}}}$, should be equal to the sum of the conveyed flow, $\mathrm{QS}_{\mathrm{N}_{\mathrm{I1}},\mathrm{N}_{\mathrm{I2}}}$, from node N on INL I1 to node N on INL I2. A continuity equation is written for each node on INL N_{I1}.

   $\mathrm{Q}_{\mathrm{N}_{\mathrm{I1}}}=\sum_{N_{\mathrm{I2}}} \mathrm{QS}_{\mathrm{N}_{\mathrm{I1}},\mathrm{N}_{\mathrm{I2}}}\mathrm{x}_{\mathrm{N}_{\mathrm{I1}},\mathrm{N}_{\mathrm{I2}}} \ \forall N_{I1}$ (5-2)

9. The difference between the sum of the total collected inflow, $\mathrm{QS}_{\mathrm{N}_{\mathrm{I1}},\mathrm{N}_{\mathrm{I2}}}$, at node N on INL I1 minus the sum of the total outflow to node N on INL I3 has to be equal, $\mathrm{QIN}_{\mathrm{N}_{\mathrm{I2}}}$, (it is equal to zero if it is a collection node or has a certain value if it is a source node). The continuity equation for each node N on INL $N_{I2}$is written as

$\sum_{N_{\mathrm{I1}}} \mathrm{QS}_{\mathrm{N}_{\mathrm{I1}},\mathrm{N}_{\mathrm{I2}}}\mathrm{x}_{\mathrm{N}_{\mathrm{I1}},\mathrm{N}_{\mathrm{I2}}} - \sum_{N_{\mathrm{I3}}} \mathrm{QC}_{\mathrm{N}_{\mathrm{I2}},\mathrm{N}_{\mathrm{I3}}}\mathrm{y}_{\mathrm{N}_{\mathrm{I2}},\mathrm{N}_{\mathrm{I3}}}= \mathrm{QIN}_{\mathrm{N}_{\mathrm{I2}}} \ \forall N_{I2}$ (5-3)

10. The difference between the sum of the total collected inflow, $\mathrm{QC}_{\mathrm{N}_{\mathrm{I2}},\mathrm{N}_{\mathrm{I3}}}$, at node N on INL I2 minus the sum of the total outflow to node N on INL I4 has to be equal to $\mathrm{QIN}_{\mathrm{N}_{\mathrm{I3}}}$, and it must equal zero if it is a collection node or have a certain value if it is a source node. The continuity equation for each node N on INL N_{I3} is written as

$\sum_{N_{\mathrm{I2}}} \mathrm{QC}_{\mathrm{N}_{\mathrm{I2}},\mathrm{N}_{\mathrm{I3}}}\mathrm{y}_{\mathrm{N}_{\mathrm{I2}},\mathrm{N}_{\mathrm{I3}}} - \sum_{\mathrm{N}_{\mathrm{I4}}} \mathrm{QO}_{\mathrm{N}_{\mathrm{I3}},\mathrm{N}_{\mathrm{I4}}}\mathrm{z}_{\mathrm{N}_{\mathrm{I3}},\mathrm{N}_{\mathrm{I4}}}= \mathrm{QIN}_{\mathrm{N}_{\mathrm{I3}}} \forall N_{I3}$ (5-4)

11. The sum of the conveyed flow, $\mathrm{QO}_{\mathrm{N}_{\mathrm{I3}},\mathrm{N}_{\mathrm{I4}}}$, from node N on INL I3 to node N on INL I4 (drainage node), has to be equal to the outflow (drainage design flow) at each node N on INL I4. The continuity equation for $N_{I4}$is written as

$\sum_{\mathrm{N}_{\mathrm{I3}}} \mathrm{QO}_{\mathrm{N}_{\mathrm{I3}},\mathrm{N}_{\mathrm{I4}}}\mathrm{z}_{\mathrm{N}_{\mathrm{I3}},\mathrm{N}_{\mathrm{I4}}} = \mathrm{Qd}_{\mathrm{N}_{\mathrm{I4}}} \qquad \forall N_{I4}$ (5-5)

Sewer layout constraint

12. The flow from each node N on INL I1 (N_{I1}) must flow through one node on INL I2 (N_{I2}), which can be satisfied as follows, using 0/1 binary variable $x_{N_{I1},N_{I2}}$:

$$\sum_{N_{I2}} x_{N_{I1},N_{I2}} = 1 \qquad \forall\, N_{I1} \tag{5-6}$$

13. The flow from each node N on INL I2 (N_{I2}) must flow through one node on INL I3 (N_{I3}), which can be satisfied as follows, using 0/1 binary variable $y_{N_{I2},N_{I3}}$:

$$\sum_{N_{I3}} y_{N_{I2},N_{I3}} = \begin{cases} 1 & \text{If } \sum_{N_{I3}} QC_{N_{I2},N_{I3}} > 0 \\ 0 & \text{If } \sum_{N_{I3}} QC_{N_{I2},N_{I3}} = 0 \end{cases} \qquad \forall\, N_{I2} \tag{5-7}$$

14. The flow from each node N on INL I3 (N_{I3}) must flow through one node on INL I4 (N_{I4}), which can be satisfied as follows, using 0/1 binary variable $z_{N_{I3},N_{I4}}$:

$$\sum_{N_{I4}} z_{N_{I3},N_{I4}} = \begin{cases} 1 & \text{If } \sum_{N_{I4}} QO_{N_{I3},N_{I4}} > 0 \\ 0 & \text{If } \sum_{14} QO_{N_{I3},N_{I4}} = 0 \end{cases} \qquad \forall\, N_{I3} \tag{5-8}$$

15. Upper and lower bound Qmax and Qmin.

$$Qmin_{iN_{I1},N_{I2}} x_{N_{I1},N_{I2}} \leq QS_{N_{I1},N_{I2}} \leq Qmax_{N_{I1},N_{I2}} x_{N_{I1},N_{I2}} \qquad \forall\, (N_{I1}, N_{I2}) \tag{5-9}$$

$$Qmin_{N_{I2},N_{I3}} y_{N_{I2},N_{I3}} \leq QC_{N_{I2},N_{I3}} \leq Qmax_{N_{I2},N_{I3}} y_{N_{I2},N_{I3}} \qquad \forall\, (N_{I2}, N_{I3}) \tag{5-10}$$

$$Qmin_{N_{I3},N_{I4}} z_{N_{I3},N_{I4}} \leq QO_{N_{I3},N_{I4}} \leq Qmax_{N_{I3},N_{I4}} z_{N_{I3},N_{I4}} \qquad \forall\, (N_{I3}, N_{I4}) \tag{5-11}$$

5.2.2 Example Application 1

This example is a theoretical system to check whether the system works in the GAMS model. Here, the location of source nodes are fixed nodes:

- Three source nodes on INL I1 (n1, n2, n3).

- One source node on INL I2 (n9).
- One source node on INL I3 (n8).

For collection nodes:

- Two candidate locations for collection nodes on INL I2 (n4, n6).
- One candidate location for a collection node on INL I3 (n7).

For drainage nodes:

- Two candidate locations for flows at drainage nodes on INL I4 (n9, n10).

The inflows at sources are found in Figure 5.3. The costs are assumed to be \$1.2 /flow for a sewer connection and \$3.2/drainage flow. The capacity limitation of flow at drainage is not included in this example.

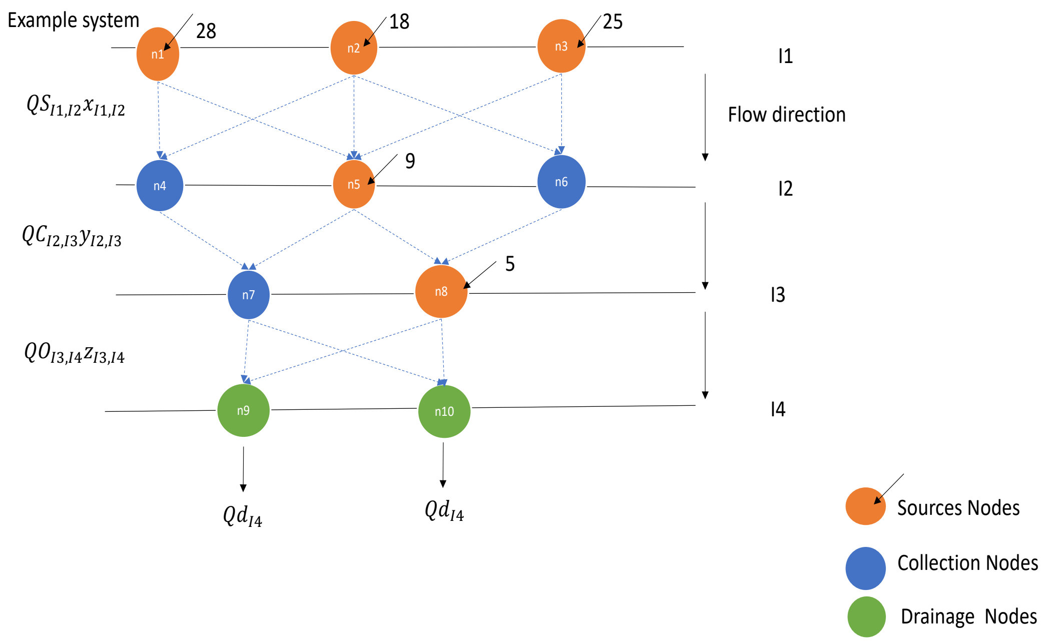


Figure 5.2. Input Data for Example Application 1.

5.2.2.1 Results

The result shows that flow is taken from each INL to INL I4 (drainage nodes). The original idea of this example was to find a minimum cost for the layout of the system and location of flows at drainage points. As shown in Figure 5.4, the layout of the system is very likely to be a dendritic type network, which is considered to only have connections between nodes and location of flows at drainage points. Further, it is preferred that the flows are collected at collections nodes more than at source nodes, because the costs would be less expensive than collecting all inflows in a certain node and then delivered to flows at drainage nodes. The cost functions are a function of flow, so the total costs for this system is $555,200.

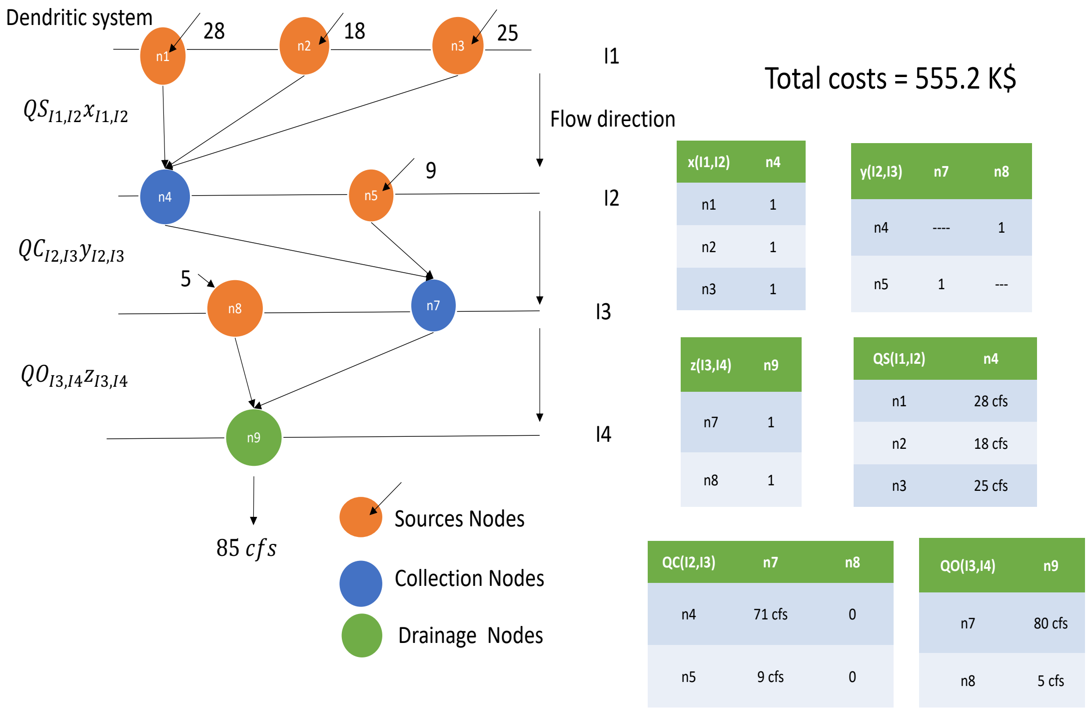


Figure 5.3. Results of Example Application 1.

5.2.3 Example Application 2

This example is also a theoretical system that is an expansion of first example. The idea here was to develop another model in GAMS program by introducing flow, QP, and a binary variable, r, that is connected to INL 4 and INL 5. Here, the location of source nodes are fixed nodes:

- Four source nodes on INL I1 (n1, n2, n3, n4).
- One source node on INL I2 (n6).
- Two source nodes on INL I3 (n10, n11).
- Two source nodes on INL I4 (n12, n13).

For collection nodes:

- Three candidate locations for collection nodes on INL I2 (n5, n7, n8).
- One candidate location for the collection node on INL I3 (n9).
- One candidate location for a collection node on INL I4 (n14).

For drainage nodes:

- Three candidate locations for drainage nodes on INL I5 (n15, n16, n17).

The inflows at sources are shown in Figure 5.5. The costs were assumed to be \$1.2/flow for sewer connection and \$3.2 for drainage flow. The capacity limitation of drainage areas is not included in this example.

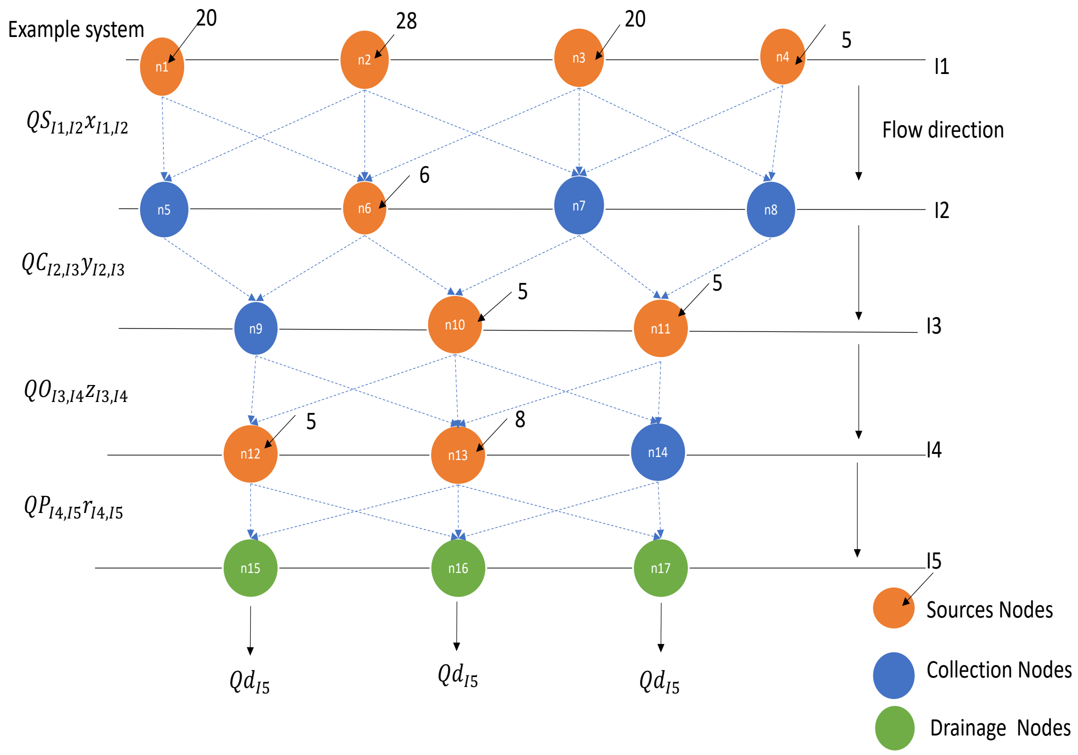


Figure 5.4. Input Data for Example Application 2.

5.2.3.1 Results

The results show that flow is taken from each INL to INL I5. As shown in Figure 5.6, it is the same as previous example: the system design can be a dendritic type network. The total costs for this system are $738,000, which is higher than the first example. This is because the total flow in this example is higher than first one (85 cfs ~ 102 cfs).

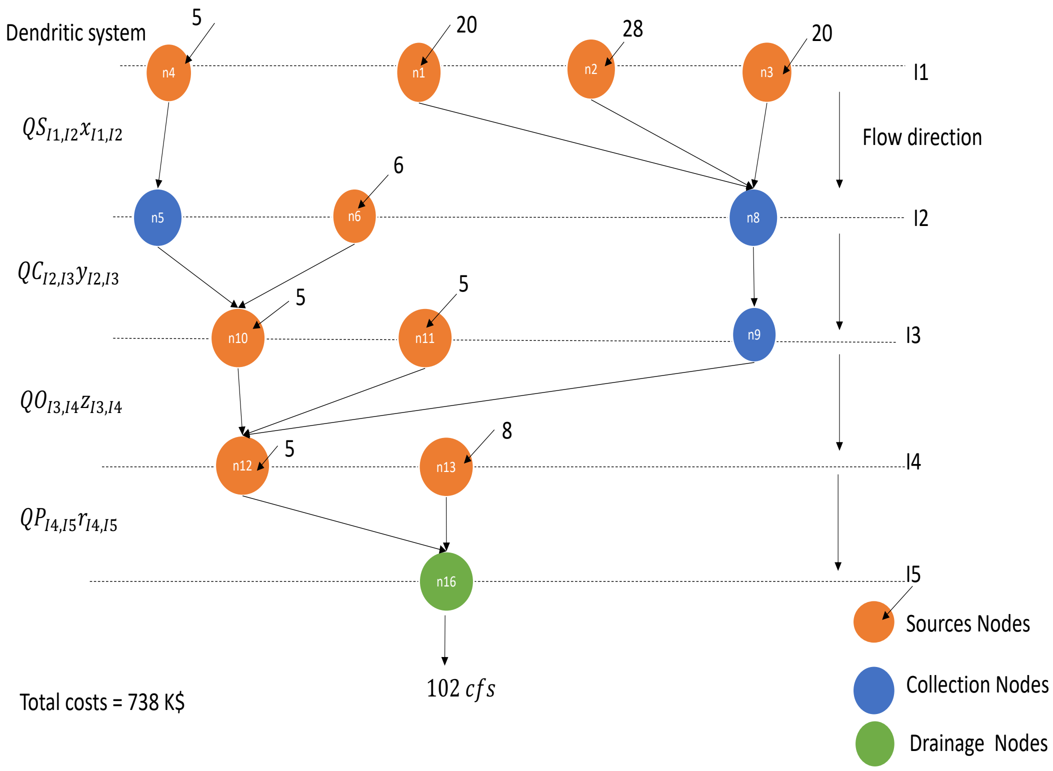


Figure 5.5. Results of Example Application 2.

6 APPLICATION OF OPTIMIZATION MODELS FOR PLANNING REGIONAL WASTEWATER SYSTEMS: CASE STUDY: JIZAN REGION, KSA

6.1 Introduction

The objectives of this chapter are 1) to improve the layout design model developed in Chapter 4 for regional wastewater system, considering the total costs of sewer networks and WWTPs; 2) to apply the model to Jizan region, KSA, which has 34 cities with populations over 5,000 people; and 3) to evaluate the current strategic plan for the wastewater system in the region.

6.2 Previous Optimization Models

A detailed survey of the models presented in the literature during the early 1960s to the 1990s is presented in de Melo and Camara (1994). Recent work on optimization models for planning regional wastewater systems as combined systems (sewer network, pump station, and WWTPs) is offered by Zeferino et al. (2014). Zeferino et al. (2017) used a single objective function to maximize water quality or minimize costs for a regional wastewater system. Applying an SA algorithm to a case study in the Una River Basin region in Brazil, Brand and Ostfeld (2011) developed a model focused on developing the cost functions of regional waste water system components using a GA. Al-Zahrani et al. (2016) used a multi-objective goal programming approach to simulate water distribution from multiple sources to multiple users in Riyadh, KSA over a 35-year period. More information about previous optimization models is provided in Chapter 2.

6.3 Mathematical Formulation

The optimization model, referred to as M-1 to distinguish it from other models, treats wastewater (generated at source nodes) at WWTP nodes, therefore allowing us to achieve minimum costs. The model is developed to consider minimum costs, which will help decide the optimal city to allocate the WWTP. The objective function considers the discounted costs of transportation and the WWTP itself. Figure 6.1 shows an example of a simple layout system provided in Chapter 4 for regional wastewater systems, which considers source nodes i, candidate locations of collection nodes j, and candidate locations of WWTPs nodes k.

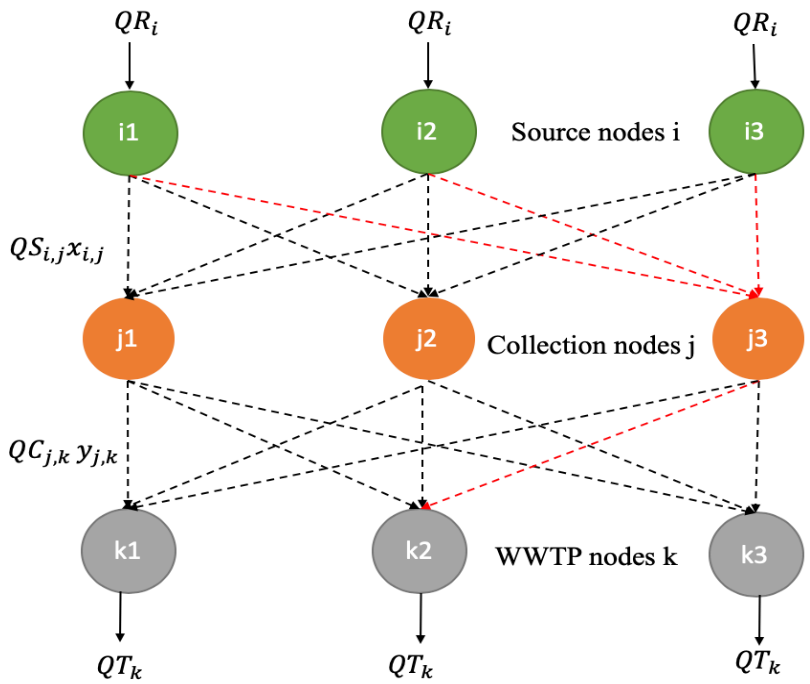


Figure 6.1. Example System Layout

The model objective function minimizes the total costs associated with the construction, operation, and maintenance of a sewer system and WWTP by determining the optimal layout design of the sewer pipes' network and WWTP(s).

$$\text{Min Cost} = \sum_i \sum_j C_{Sewer(i,j)}\, QS_{i,j}\, L_{i,j}\, x_{i,j} + \sum_j \sum_k C_{Sewer(j,k)} QC_{j,k}\ \ L_{j,k}\, y_{j,k} + \ldots + \sum_k C_{WWTP,k}(QT_k) \quad (6\text{-}1)$$

Where

$C_{Sewer(i,j)}$ is the discounted cost of the installation, operation, and maintenance of sewer system from nodes on INL i to nodes on INL j.

$QS_{i,j}$ the flow rate from nodes on INL i to nodes on INL j.

$L_{i,j}$: the length of pipe between nodes on INL i to nodes on INL j

$x_{i,j}$: 0/1 binary variable that will take value 1 if there is an existence of a particular pathway that links node i to node j and 0 otherwise.

$C_{Sewer(j,k)}$: the discounted cost of installation, operation, and maintenance of sewer system from nodes on INL j to outlet (WWTP) nodes on INL k.

$QC_{j,k}$: the flow rate from nodes on INL j to outlet (WWTP) nodes on INL k.

$L_{j,k}$: the length of pipe between nodes on INL j to outlet (WWTP) nodes on INL k.

$y_{j,k}$: 0/1 binary variable that will take value 1 if there is an existence of a particular pathway that links node j to node k and 0 otherwise.

$C_{WWTP,k}$: the discounted cost of treated wastewater at WWTP node k.

QT_k: the flow rate of treated wastewater at WWTP node k.

Subject to:

Continuity constraint

16. The produce flow, QR_i, at source node i should be equal to the sum of the conveyed flow, $QS_{i,j}$, from source node i to collection node j. A continuity equation was written for each source node i.

$$QR_i = \sum_j QS_{i,j} x_{i,j} \qquad \forall\, i \qquad (6\text{-}2)$$

17. The difference between the sum of the total collected inflow, $QS_{i,j}$, at collection node j minus the sum of the total outflow to wastewater treatment plan k have to be equal to zero. The continuity equation for each collection node j is written as

$$\sum_i QS_{i,j}x_{i,j} - \sum_k QC_{j,k}\, y_{j,k}=0 \qquad \forall\, j \qquad (6\text{-}3)$$

18. The sum of the conveyed flow, $QC_{i,j}$, from collection node j to WWTP node k, have to be equal to outflow (treated wastewater) at each WWTP node. The continuity equation for each WWTP node is written as

$$\sum_J QC_{j,k}\, y_{j,k} = QT_k \qquad \forall\, k \qquad (6\text{-}4)$$

19. The sum of the total produced wastewater, QR_i, at source nodes i=1,2,….,I, should be equal to the sum of wastewater treated, QT_k, at wastewater treatment nodes k=1,2,…, K, as follows: The wastewater that produced at source nodes should be treated.

$$\sum_i QR_i = \sum_k QT_k \qquad (6\text{-}5)$$

20. The sum of the conveyed flow, $QC_{i,j}$, from collection node j to WWTP node k, have to be equal or less than maximum WWTP capacity, MaxQT,. The capacity equation for this constraint would be written as following.

$$\sum_j QC_{j,k}y_{j,k} \leq MaxQ_{wwtp} \qquad \forall\, k \qquad (6\text{-}6)$$

Connectivity constraint

21. The flow from each source node i must flow through one collection node j, which can be satisfied as follows using 0/1 binary variable, $x_{i,j}$,:

$$\sum_j x_{i,j} = 1 \qquad \forall\, i \qquad (6\text{-}7)$$

22. The flow from each collection node j must flow through one WWTP node k, which can be satisfied as follows using 0/1 binary variable $x_{i,j}$:

$$\sum_k y_{j,k} = \begin{cases} 1 & \text{If } \sum_k QC_{j,k} > 0 \\ 0 & \text{If } \sum_k QC_{j,k} = 0 \end{cases} \qquad \forall\, j \qquad (6\text{-}8)$$

- The flow through the sewer system should be between maximum and minimum flows, which can be satisfied as follows using Qmax and Qmin.

$$Qmin_{i,j} x_{i,j} \leq QS_{i,j} \leq Qmax_{i,j} x_{i,j} \qquad \forall\, (i, j) \qquad (6\text{-}9)$$

$$Qmin_{j,k} y_{j,k} \leq QC_{j,k} \leq Qmax_{j,k} y_{j,k} \qquad \forall\, (j, k) \qquad (6\text{-}10)$$

The model was developed in GAMS using the MINLP solver. The model used several case scenarios as presented in Chapter 4.

6.4 Case Study Application

Jizan region, KSA was chosen as the case study in this dissertation. According to GAS (2017), there were 34 cities in the region with a population of over 5,000 people in 2017 (https://www.stats.gov.sa/en). These cities are wastewater sources with a total population of 1.3 million people and a daily wastewater generation rate per capita of 0.01 m^3 to 0.213 m^3. The wastewater generation for each city is calculated by introducing many factors, such as population, average water consumption, observed water demand, growth rate, water supply, losses and infiltration, leakage factor, wastewater network coverage, and safety factors. Table 6.1 shows the summary of the data used in this study. The distance between any two cities is provided in the grid network for each zone. The detailed calculation of wastewater generated for each city is provided in Appendix B.

Table 6.1. Summary of Data.

No#	Cities	Node number	Wastewater generated (m^3/day)	Elevations (m)
1	Jazan	n1	25673	0
2	Sabya	n2	33355	30
3	Abu Arish	n3	29074	70
4	Samtah	n4	18539	70
5	Damad	n5	7863	80
6	Al Darb	n6	6037	80
7	Addayer	n7	6453	800
8	Ahad Al Masarihah	n8	16189	80
9	Baish	n9	5447	80
10	Alaliya	n10	4248	50
11	Al Shugayri	n11	3040	110
12	Al Fatiha	n12	1058	220
13	Qawz al Ja’afirah	n13	3022	0
14	Al Kadami	n14	2692	100
15	Wadi Jizan	n15	7901	30
16	AlGofol	n16	3739	120
17	Al Sehi	n17	3236	0
18	Al Khubah/Alharth	n18	1257	250
19	Khushal	n19	1543	250
20	Al Qasabah	n20	2754	250
21	Al Reeth	n21	2154	800
22	Al Haqu	n22	1546	200
23	Masliyah	n23	2719	120
24	Alaydabi	n24	1156	230
25	Al Madaya	n25	3876	10
26	Al Aridhah	n26	6922	200
27	Alhumira	n27	1488	230
28	Al Shuqaiq	n28	3597	10
29	Al Tuwal	n29	5725	70
30	Harub	n30	2401	450
31	Fayfa	n31	4489	1200
32	Itwide	n32	766	30
33	Aiban/Belghazi	n33	3625	400
34	Al Mwassam	n34	2679	15

The method for defining the potential connect network was to link each node to all neighboring nodes that could be reached via gravity flow. The case study is divided into five different zones based on topography, size of wastewater produced, the importance of assumed candidate locations of WWTPs, and administration boundaries. Each zone contains a couple of cities and a candidate location for collection and/or WWTPs. There are trade-offs between transportation costs and WWTP costs. Separate optimization codes are developed for each zone to find the optimal location, type, and size of WWTPs and gravity sewage network. Figure 6.2 provides the terrain map of the case study region, showing that ground elevations rise toward the east.

In the Jizan region there is only one existing WWTP (W-1) at Zone 4. The government is currently constructing treatment plants in several cities and is planning to construct plants in other cities. Table 6.2 shows the list of existing, under construction, and planned treatment plants in the region (MEWA, 2016).

Table 6.2. List of Existing, under Construction and Planned Treatment Plants in the Region (MEWA, 2016).

City	Capacities (m^3/day)					Total capacity (m^3/day)
	Existing 2011	Under construction 2015	Planned 2020	Planned 2025	Planned 2030-2035	
Jazan	20,000	33,000	19,000	-	-	64,000
Sabya	-	33,000	11,000	-	-	44,000
Abu 'Arish	-	30,000	-	-	-	30,000
Samtah	-	3,000	-	12,000	-	15,000
Baysh	-	-	16,000	-	-	16,000
Ahad Al Masarihah	-	-	12,000	-	-	12,000
Damad	-	-	16,000	-	-	16,000
Ad Drab	-	-	5,000	-	-	5,000
Al 'Aliyah	-	-	4,000	-	-	4,000
Ad Dai'r	-	-	4,000	-	-	4,000
Al Shuqairy	-	-	3,250	-	-	3,250
Farasan	-	-	3,500	-	-	3,500

The cost functions for regional wastewater systems, including installation, maintenance, and operation costs, are non-linear functions. Brand and Ostfeld (2011), Mays et al. (1983), and Olej et al. (2011) presented cost functions for regional water/wastewater systems, including installation, operation, and maintenance costs. These functions are strictly non-linear equations and are hard to define for different regions and economies of scale. This indicates that the solution would be to concentrate treatment into one or very few plants rather than into many plants.

The discount costs for construction, operations and maintenance for sewer pipelines and WWTP for the case study are based on data from previous projects. The sewer pipe construction costs include pipe costs and excavation costs. The cost values of estimated sewer material pipe costs were around \$300/m for vitrified clay (VC) less than 600 mm, which includes all appurtenances. Therefore, the excavation costs, including all earthwork, are estimated to be \$3 m^3/m, and the total costs per unit discharge for 1 km are estimated to be \$30 $Q\left(\frac{m^3}{day}\right)\ L(km)$. The cost values for WWTP, including size, type, and time life of treatment, are estimated by the following function:

$$\text{Costs}_{m^3/day} = 0.0101Q_T^2 + 895.9Q_T + 62838$$

All Q and Q_T is in m^3/day and L is in (km).

This information is estimated based on data derived from MEWA and previous projects in KSA. The first application of the case study would be to find the optimum planning design of the system with regards to minimum costs of installation, operation, and maintenance of gravity sewage network and WWTP using M-1. The procedure to reach a solution for this type of problem is not only to find the optimum planning design, but also to provide another alternative solution once the input data is adjusted.

6.4.1 Zone 1

Zone 1 is the smallest zone, with three cities that have to collect and treat wastewater. However, this study introduces two more cities (Al Shuqaiq and Itwide) into the system, which have populations of over 5,000 people. Figures 6.2 and 6.3 show a horizontal view for Zone 1. As shown in Table 6.1, the elevations of Ad Darb (n6), Al Shuqaiq (n28), and Itwide (n32) are 80 m, 10 m, and 30 m, respectively. However, to transfer wastewater from Al Shuqaiq to Itwide or Al Darb, pump stations must be put in place to maintain head elevations in the system. This approach will result in an addition to the energy costs of pumps in the system, the most expensive components. It is recommended to evaluate the plan by applying two potential alternative solutions in Zone 1. The first alternative solution is to have one location of the WWTP, which would change from the current plan of WWTP (W-6)'s location to Al Shuqaiq (W-28). The second alternative solution is to have two WWTPs, each in two different cities (W-6 and W-28). However, introducing a dummy node in the system is another innovative technique to explore the problem in computer code.

Figure 6.2. Horizontal View for Cities and Proposed WWTPs of Zone 1.

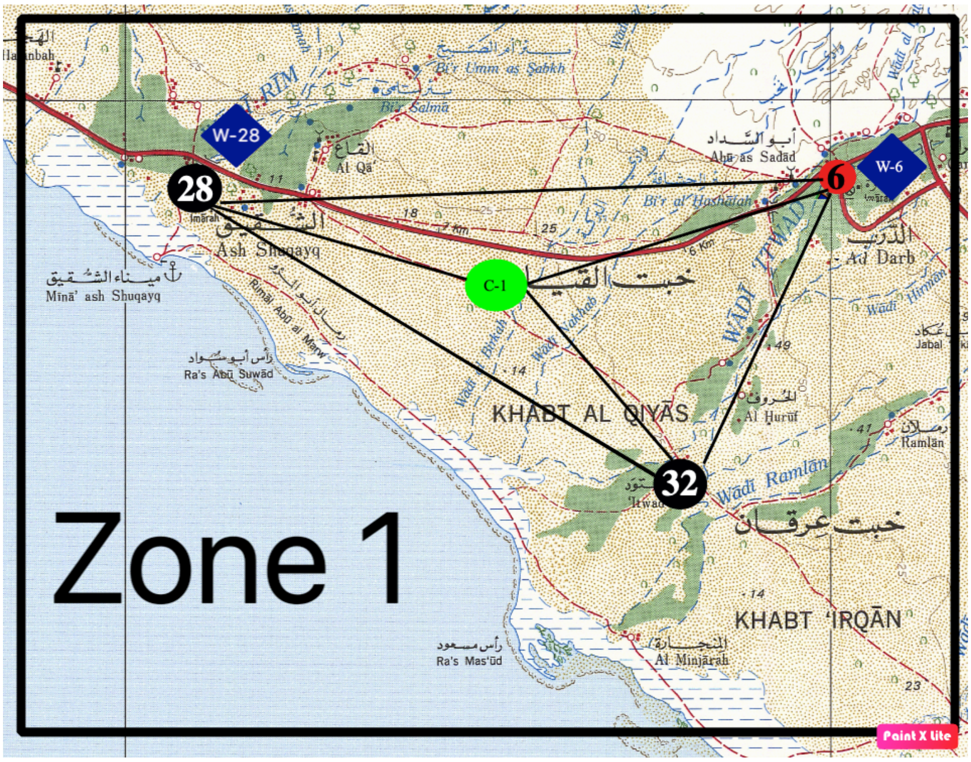


Figure 6.3. Possible Pipe Connections for Each City and Possible WWTPs of Zone 1.

6.4.1.1 Results

The grid network for Zone 1 is shown in Figure 6.4. The model results show the optimum solution for Zone 1 has two WWTPs (W-28 and W-6). The total costs of transporting wastewater flow from Ad Darb (n6) to a new WWTP (W-28) appear to be much higher than having two WWTPs (W-6 and W-28). The total estimated costs of installing a WWTP and gravity sewage network is around $10.5 million. The study proposed one more WWTP (W-28), which would be located at Al Shuqaiq, with a total capacity of 5,000 m^3/day. The planned WWTP (W-6), which is located in Ad Darb (n6), must be more than 6,000 m^3/day instead of 5,000 m^3/day.

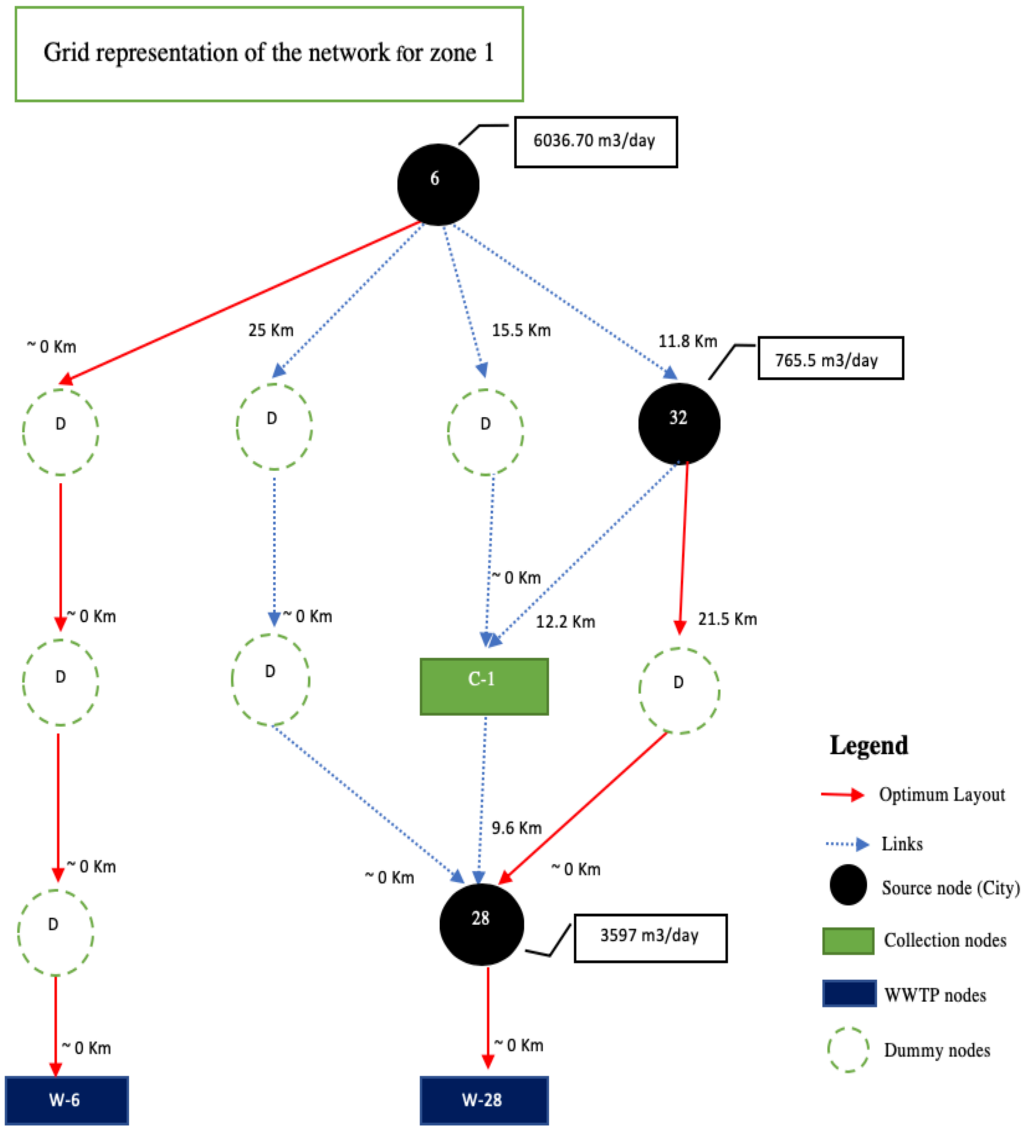


Figure 6.4. The Grid Network for Zone 1

6.4.2 Zone 2

Zone 2 considers five cities (Baish, Masliyah, Al Haqu, Al Fatiha, and Al Reeth). The proposed WWTP is located in Baish (W-9), with a total capacity of 16,000 m^3/day. Today, the operation of W-9 is a trial situation, with a total capacity of 2,000 m^3/day and reuses the treated wastewater to plant 500 trees. The target plan is to reuse treated wastewater to plant 4 million trees in the Baish region in 2020. Figure 6.5 and 6.6 provide a horizontal view of Zone 2.

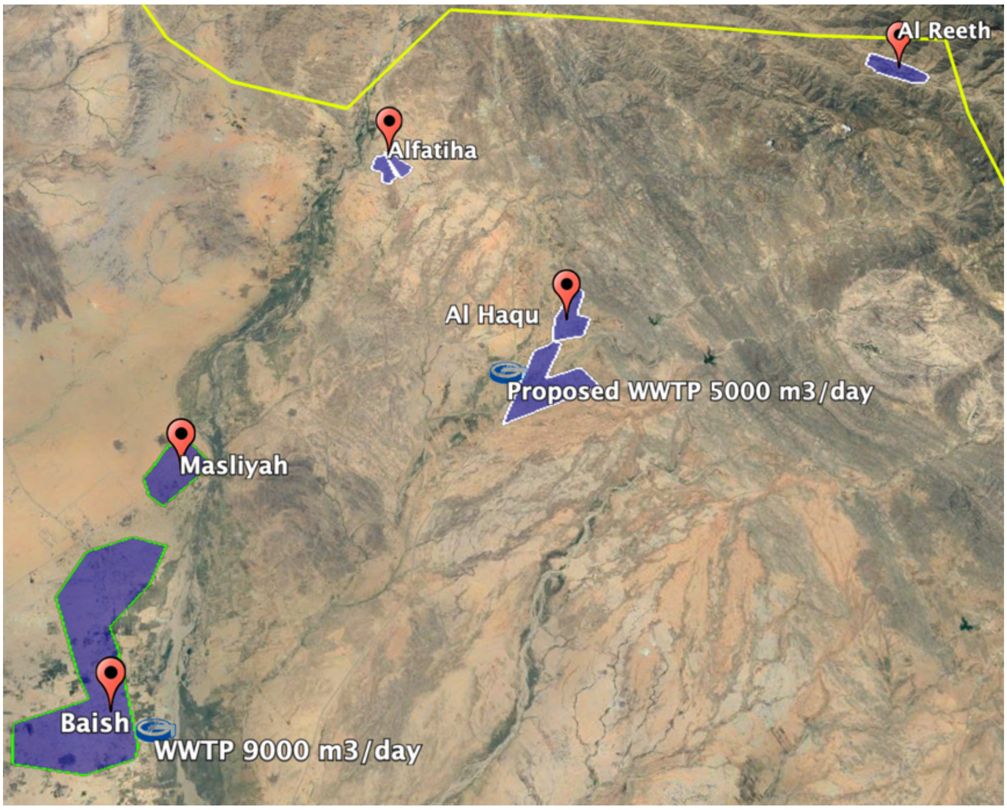


Figure 6.5. Horizontal View for Cities and Proposed WWTP for Zone 2.

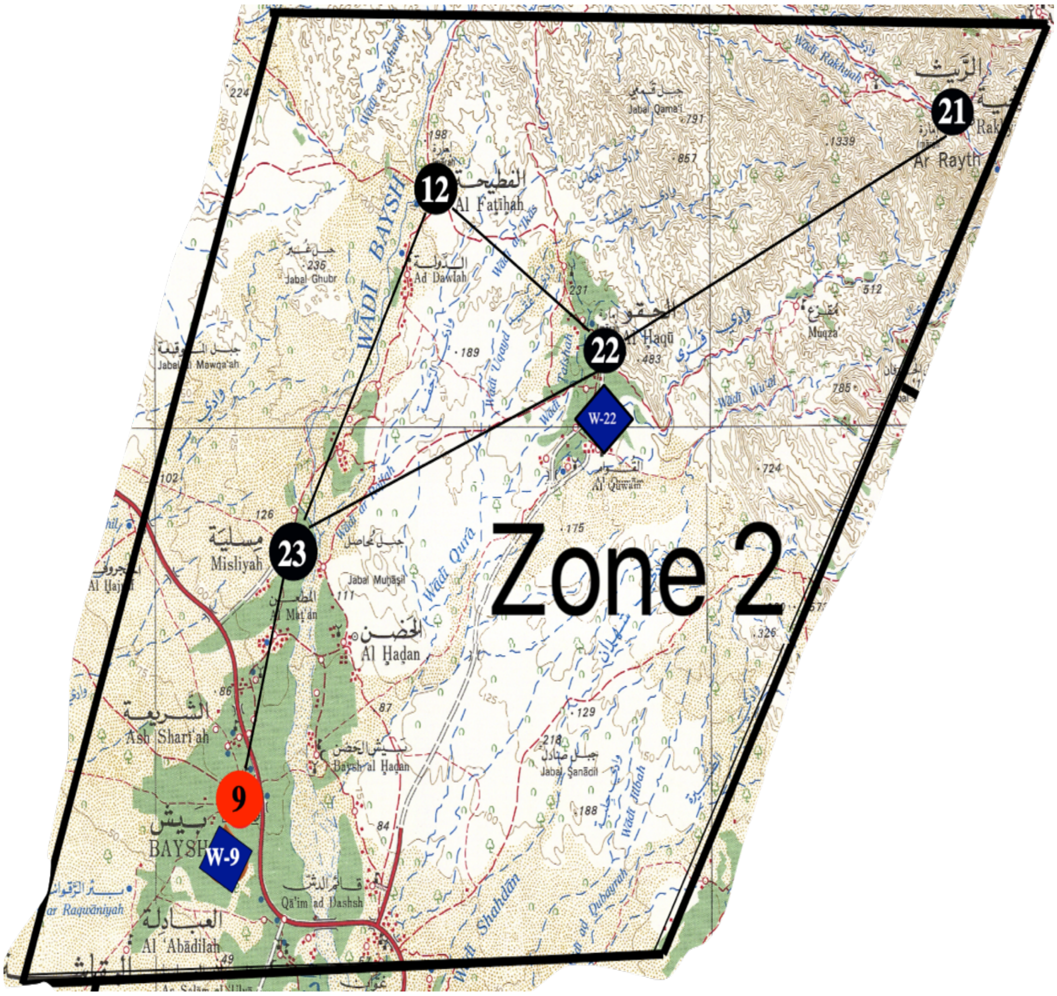


Figure 6.6. Possible Pipe Connections for Each City and Possible WWTPs of Zone 2.

The first alternative solution is to have one WWTP, allocated at Baysh (n9), so that the wastewater generated at cities (n21, n22, n12, and n23) would be transferred to one WWTP (W-9) via a gravity sewage network. This solution might be the right solution for Zone 2, but considering another alternative solution may be even better by using optimization codes. The second alternative solution is to have two WWTPs in two different cities: Baysh (n9) and Al Haqu (n22). Using the optimization model provides the answer as to whether to have one or two WWTPs based on minimizing total costs. This model assumed that the wastewater would flow via gravity between different elevations between cities and WWTP. Table 6.1 provides the summary information for all five cities in Zone 2.

6.4.2.1 Results

The grid network for Zone 2 is described in Figure 6.7. The model results show that the optimal solution for the system is having two WWTPs (W-22 and W-9). Planning two WWTPs is cheaper than transferring the generated wastewater to one WWTP. The total estimated costs of installing a gravity sewage network and WWTP is around $14 million. It proposed adding a WWTP (W-22), which would be located at Al Haqu (n22), with a total capacity of 5,000 m^3/day. The planned WWTP (W-9), which is located in Baish (n9), must treat more than 9,000 m^3/day instead of 16,000 m^3/day.

6.4.3 Zone 3

Zone 3 considers eleven cities (Sabya, Damad, Alaliya, Al Shugayri, … and Aiban/Belghazi). As shown in Figure 6.8, Zone 3 is one of the most complex systems because there are five proposed WWTPs, a huge cultivated area, with a total generated wastewater of about 70,000 m^3/day. The government planned the WWTPs (W-2, W-5, W-10, W-11, and W-7) to be located at Sabya, Damad, Alaliya, Al Shugayri, and Addayer, with total capacities or 44,000 m^3/day, 16,000 m^3/day, 4,000 m^3/day, 3,250 m^3/day, and 4,000 m^3/day, respectively. However, the study used another alternative solution to cover all cities in the region by introducing six more cities, as shown in Table 6.1 and Figure 6.8. The construction of W-2 had been 80% completed as of March 2018. However, this study proposes five WWTPs located at Sabya, Damad, Addayer, Qawz al Ja’afirah, and Harub, with 20 possible sewage network connections between cities and WWTPs. The optimization model would decide the best solution to allocate WWTPs and sewage connections based on minimum costs. Figure 6.9 shows the possible sewer network and WWTPs for Zone 3. It also shows that the authors neglected the candidate locations of WWTPs at Alaliya and Al Shugayri regarding the minimization of the number of candidate locations. A recommendation would be to connect another city, such as Qawz al Ja'afirah.

6.4.3.1 Results

The grid network for Zone 3 is described in Figure 6.10. The results of the model show that the optimum solution for the system includes five WWTPs (W-7, W-30, W-5, W-13, and W-2). As mentioned above, the planned WWTPs at Alaliya and Al Shugayri are not necessary for this zone, and other cities, such as Harub and Qawz al Ja'afirah, have been taken into consideration. The allocated WWTPs at Harub and Qawz al Ja'afirah would save costs more than transferring wastewater to a downstream candidate WWTP. The total costs for installing a gravity sewage network and WWTP is \$91 million for Zone 3. The study proposed two more WWTPs (W-13 and W-30), located at Harub (n30) and Qawz al Ja'afirah (n13), with total capacities of 3,000 m^3/day and 8,000 m^3/day, respectively. The planned WWTP (W-2), which is located in Sabya (n2), must be treat than 34,000 m^3/day instead of 44,000 m^3/day. In addition, the planned WWTP (W-5), which is located in Damad (n5), must treat more than 19,000 m^3/day instead of 16,000 m^3/day.

Furthermore, the planned WWTP (W-7) in Addayer (n7) must treat more than 11,000 m^3/day instead of 4,000 m^3/day.

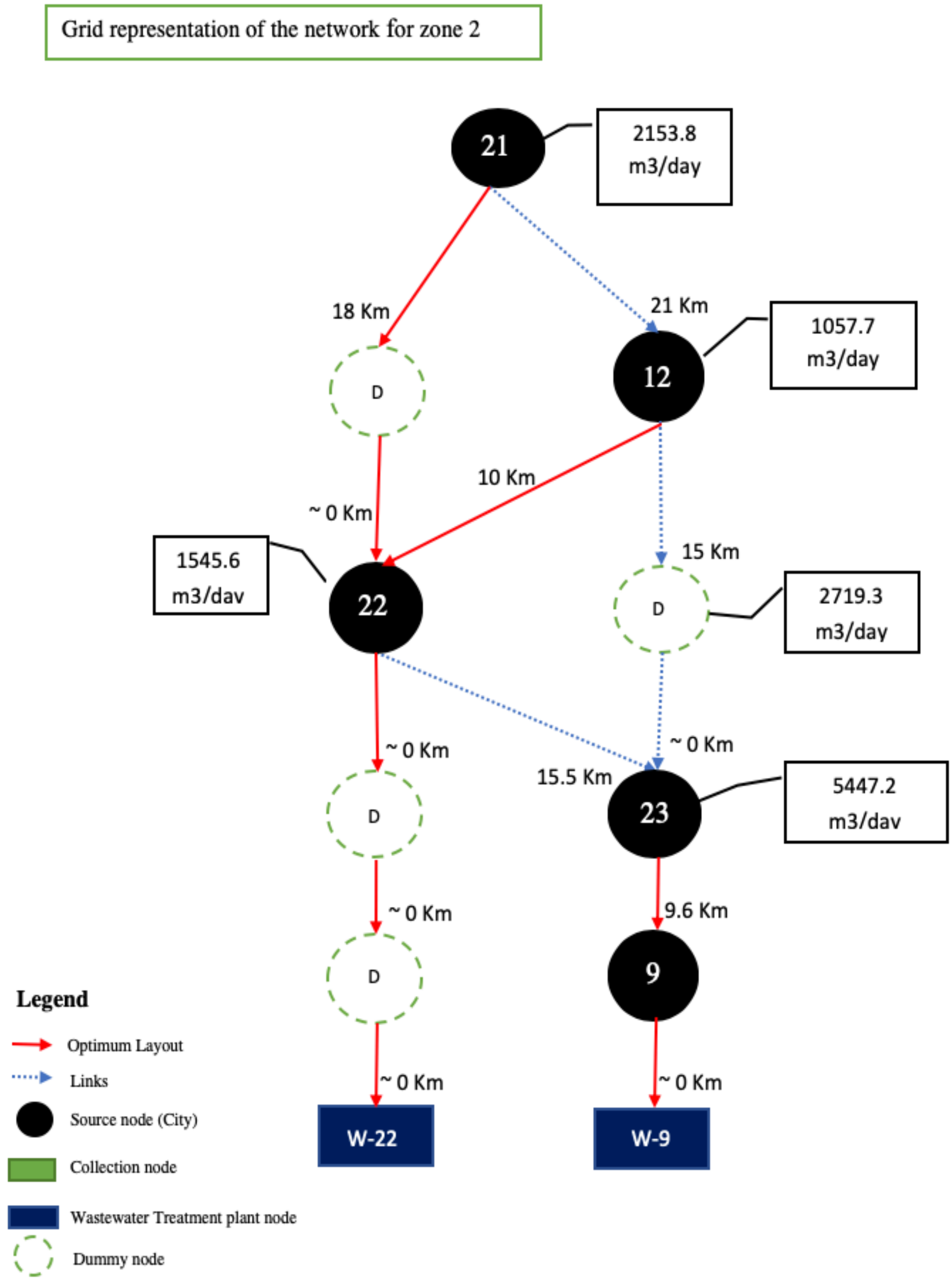


Figure 6.7. The Grid Network for Zone 2

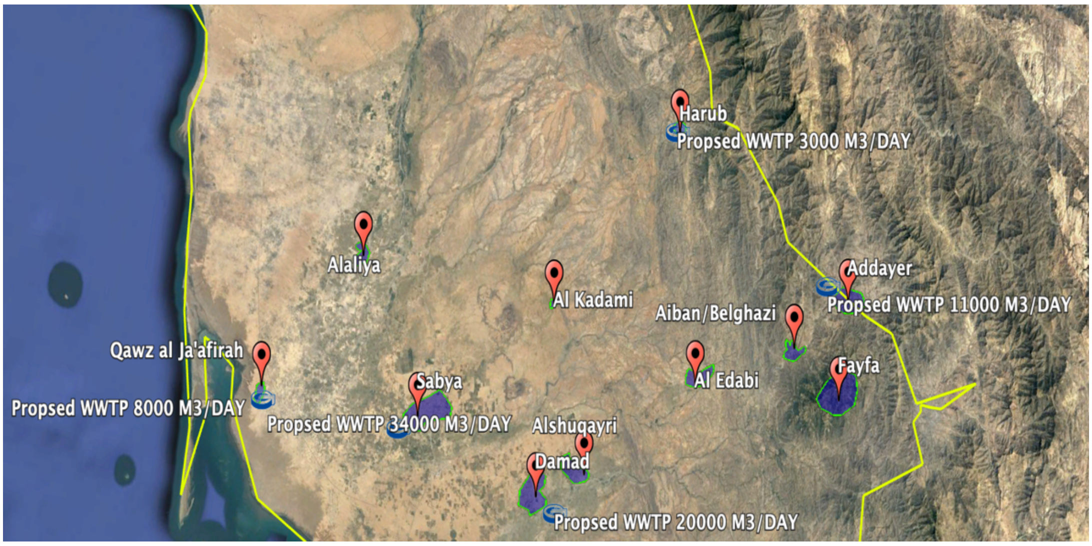


Figure 6.8. Horizontal View for Cities and Proposed WWTPs Zone 3.

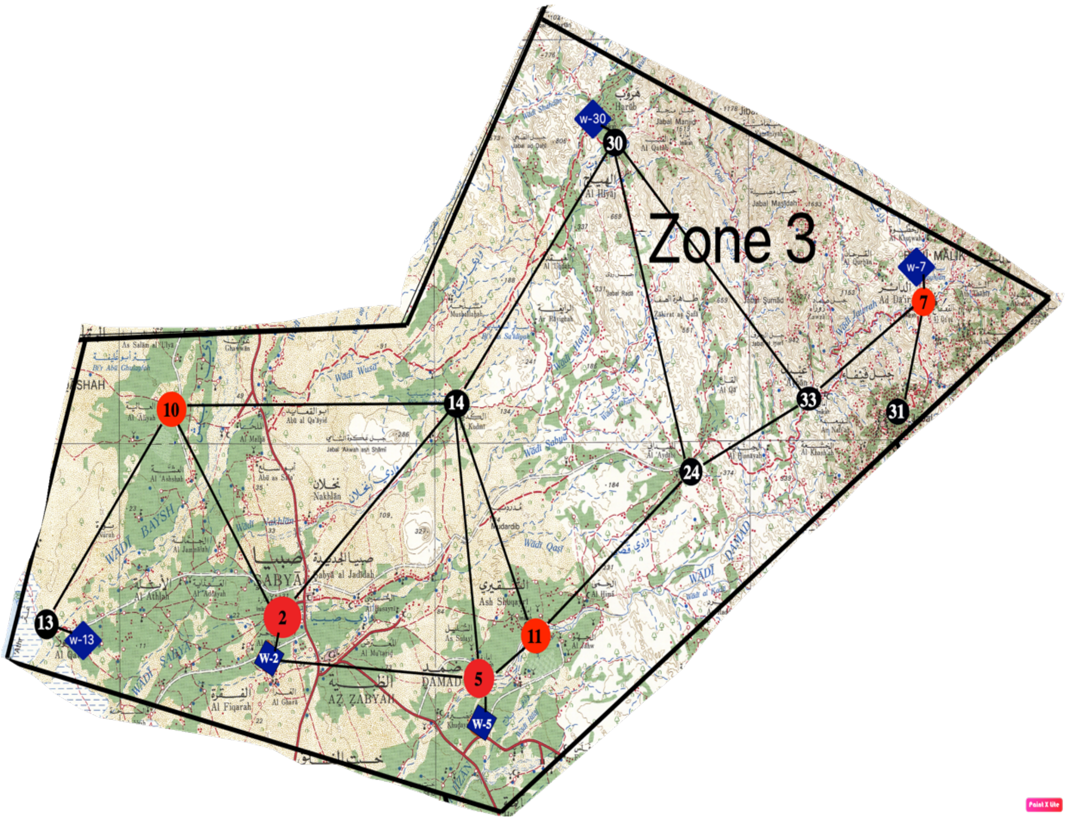


Figure 6.9. Possible Pipe Connections for Each City and Possible WWTPs of Zone 3.

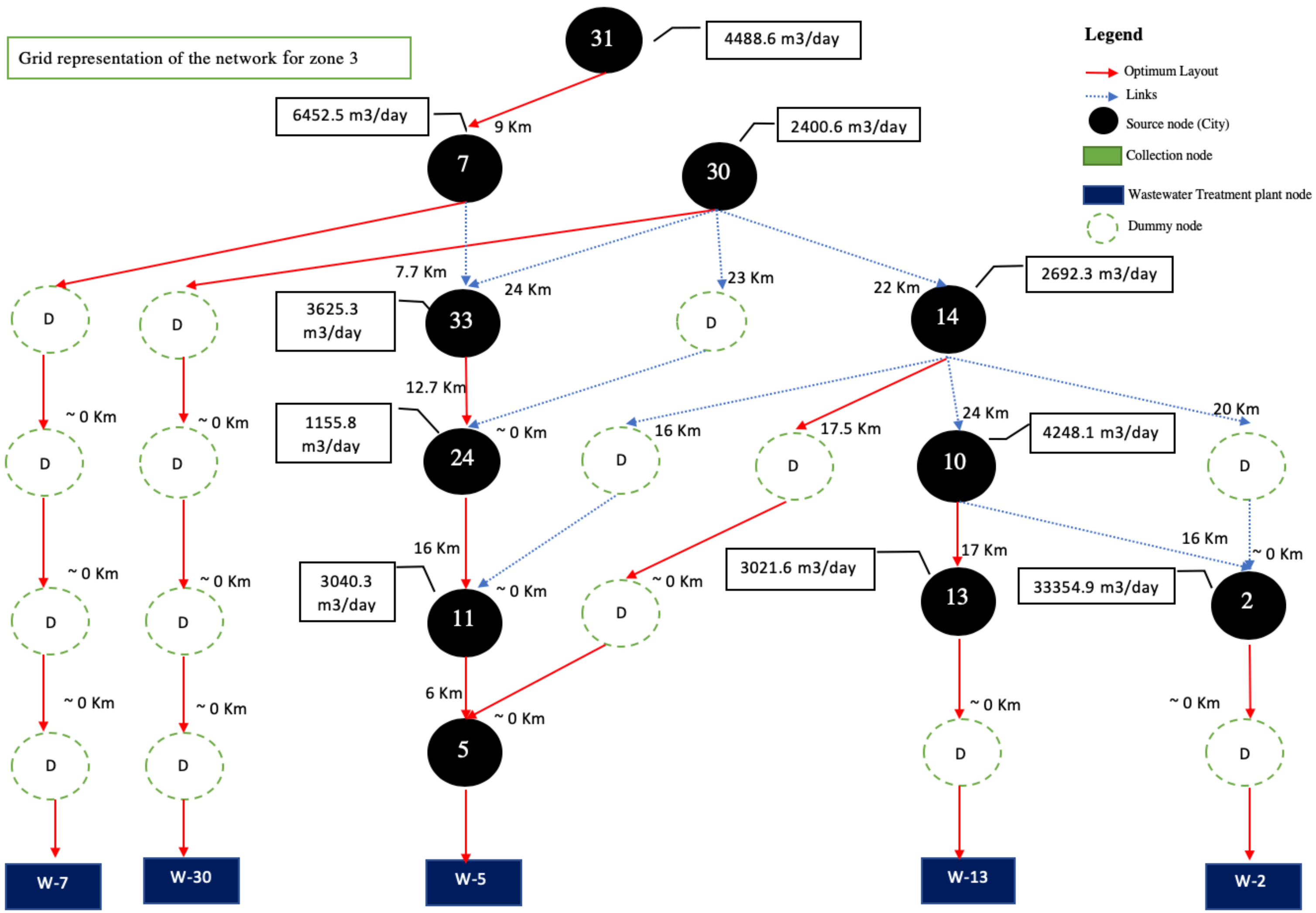


Figure 6.10. The Grid Network for Zone 3.

6.4.4 Zone 4

In Zone 4, the study considered three cities (Jazan, Wadi Jizan, and Al Madaya) and two WWTPs (W-1, W-15). The government-planned capacity of W-1 will be 64,000 m^3/day by 2020. The planned capacity seems to be large enough to cover all uncertainty in generated wastewater. However, Wadi Jizan was assumed to be one source node, which in reality is a long valley that is surrounded by 366 towns/villages (Jizan municipality website). It assumed that all towns and villages would be under one source node by taking the sum of wastewater flow and populations of these towns. Figures 6.11 and 6.12 show a horizontal view for Zone 4.

Figure 6.11. Horizontal View for Cities and Proposed WWTPs Zone 4.

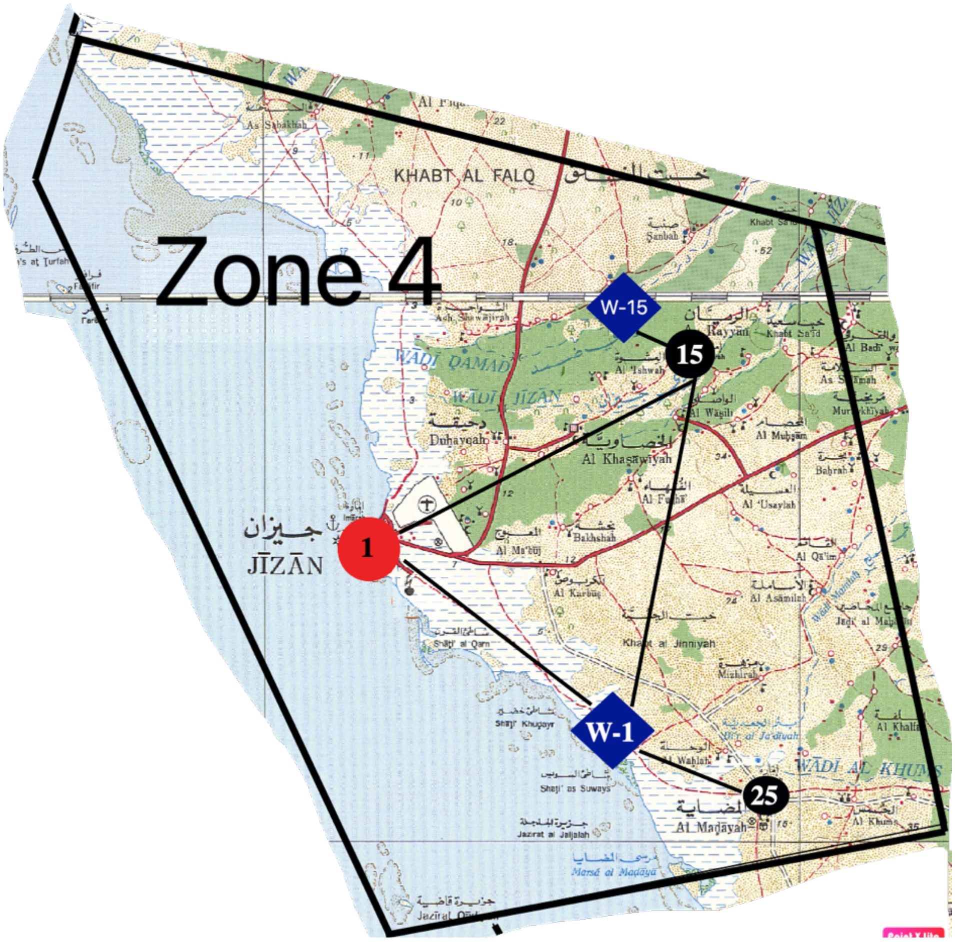


Figure 6.12. Possible Pipe Connections for Each City and Possible WWTPs of Zone 4.

6.4.4.1 Results

The grid network for Zone 4 is described in Figure 6.13. The optimal results include two WWTPs (W-1 and W-15) with a total capacity 30,000 m^3/day and 8,000 m^3/day. However, the results show that the generated wastewater at villages and towns in Wadi Jizan (n15) must be collected and transferred to W-15. The total costs for planning Zone 4 would be around $44 million for Zone 4. The study proposed one WWTP (W-15) to cover all of the Wadi Jizan cities, with a total capacity 8,000 m^3/day. Also, the study recommends a size capacity of 30,000 m^3/day instead of the enormous capacity of 64,000 m^3/day for W-1.

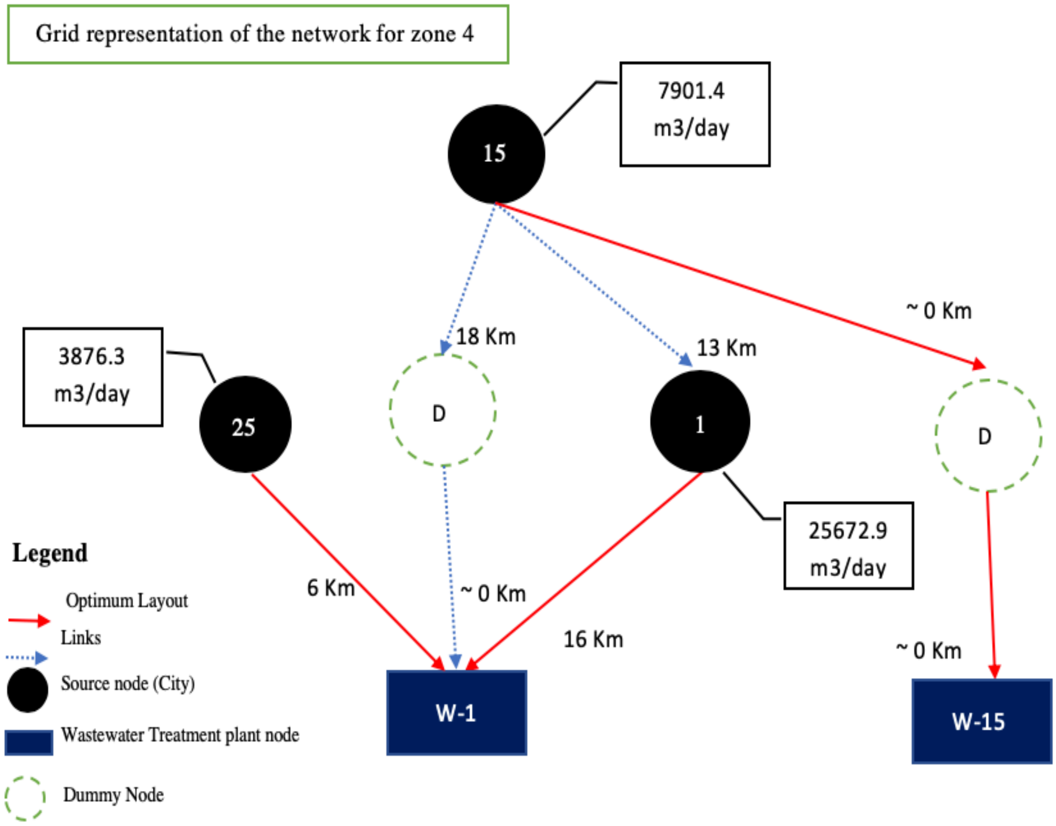


Figure 6.13. The Grid Network of Zone 4.

6.4.5 Zone 5

The total wastewater generated in Zone 5 would be around 93,000 m^3/day, which is the most significant wastewater generated in the region. Zone 5 includes 12 cities with different ranges of populations and locations. The government planned WWTPs located at three cities, Abu Arish, Samtah, and Ahad Al Masarihah, with total capacities of 30,000 m^3/day, 12,000 m^3/day, and 15,000 m^3/day, respectively. In Zone 5, there are six candidate locations for WWTPs, and there are 18 total possible sewage connections. Figures 6.14 and 6.15 show a horizontal view for Zone 5.

Figure 6.14. Horizontal View for Cities and Proposed WWTPs of Zone 5.

6.4.5.1 Results

The grid network for Zone 5 is shown in Figure 6.16. The optimal results include six WWTPs (W-3, W-8, W-4, W-17, W-26, and W-19). The total costs for installing a gravity sewage network and WWTP will be around \$110 million for Zone 5. The study proposed three WWTPs (W-17, W-26, and W-19) would be located at Al Sehi (n-17), Al Aridhah (n26), and Khushal (n19), with total capacities of 6,000 m^3/day, 9,000 m^3/day, and 5,000 m^3/day, respectively. Thee planned WWTPs W-3, W-4, and W-8, located in Abu Arish (n3), Samtah (n4), and Ahad Al Masarihah (n8), must treat 30,000 m^3/day, 25,000 m^3/day, and 22,000 m^3/day, respectively, instead of 15,000 m^3/day.

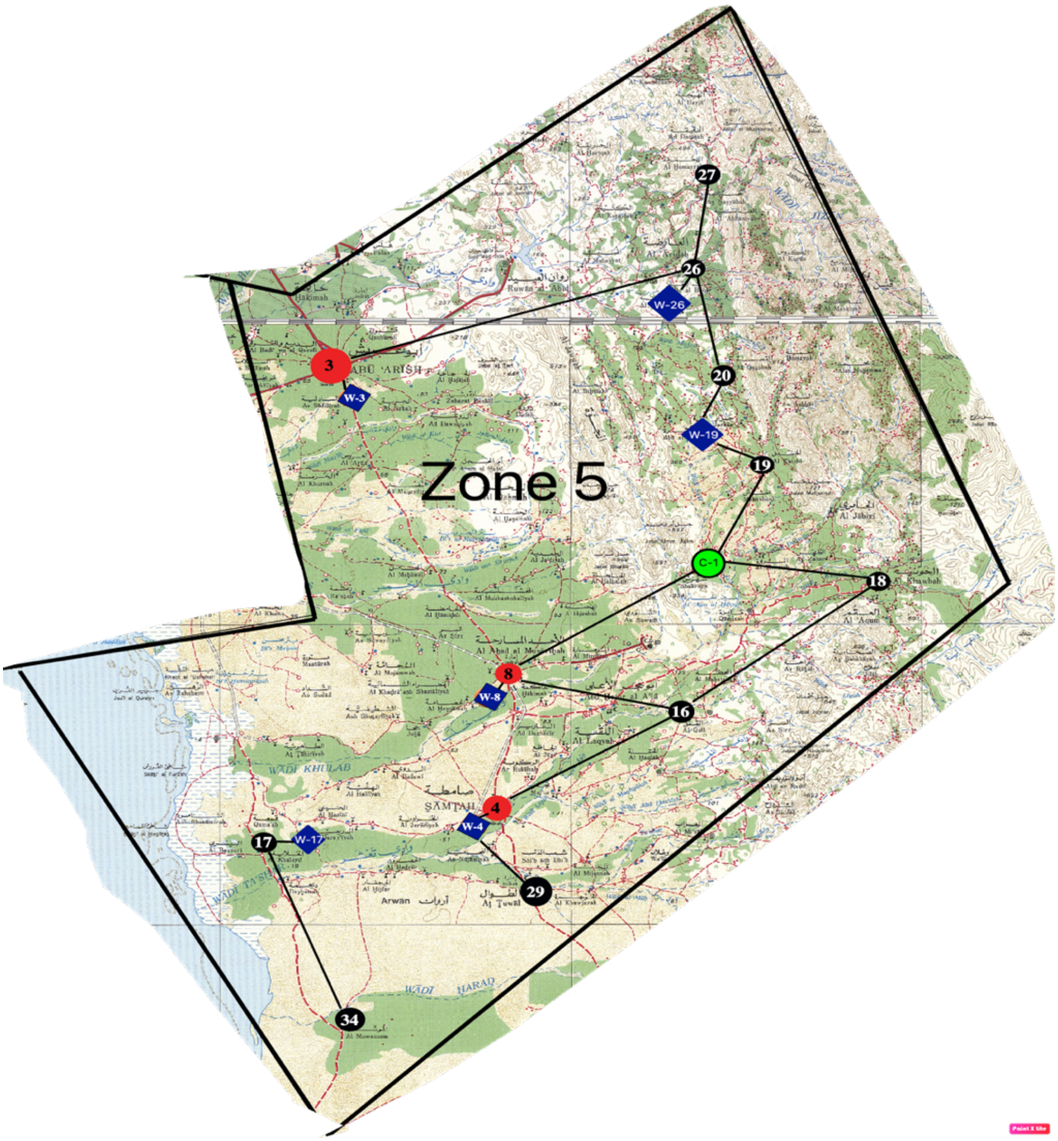


Figure 6.15. Possible Pipe Connections for Each City and Possible WWTPs in Zone 5.

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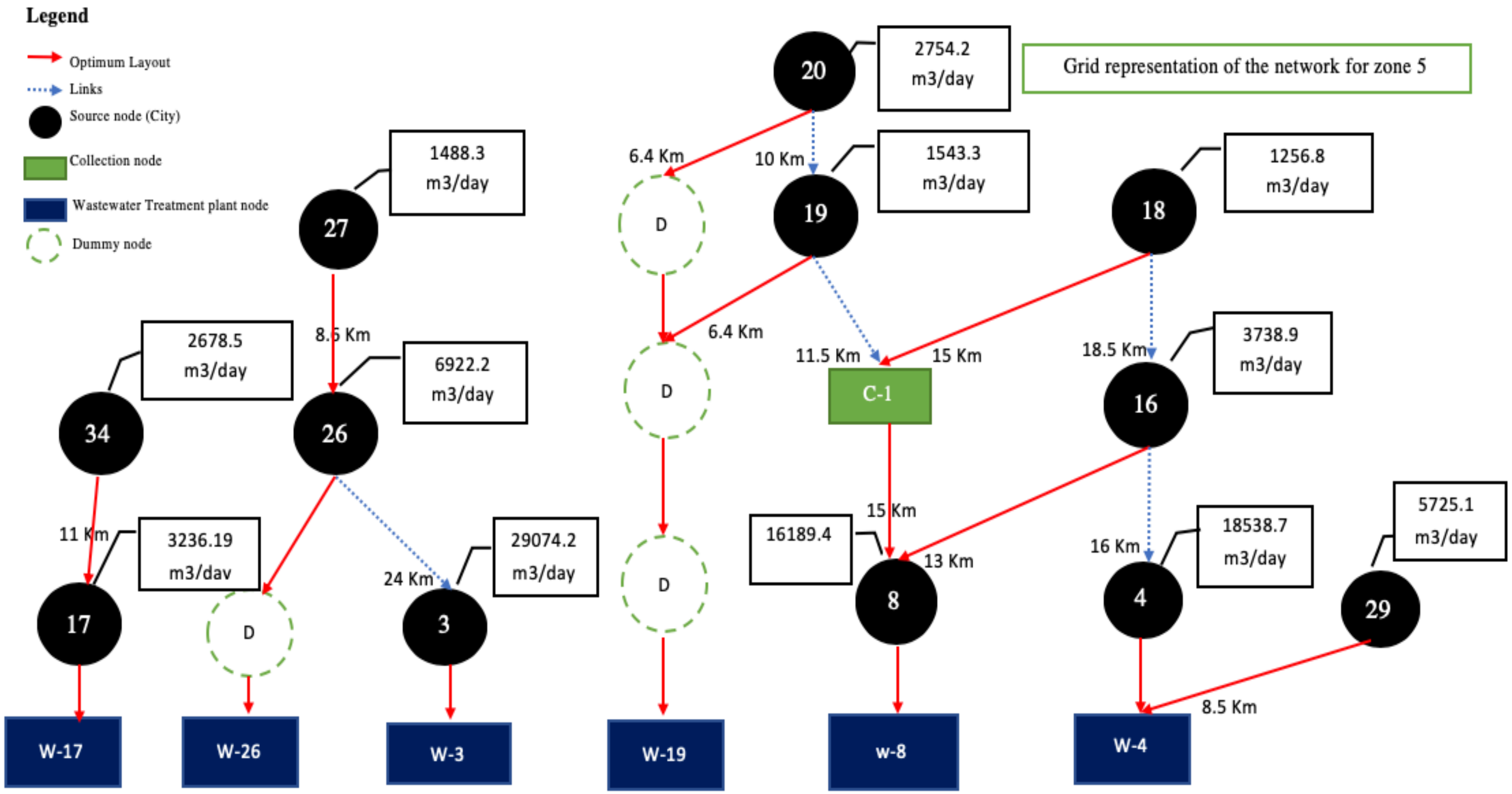


Figure 6.16. The Grid Network of Zone 5.

6.5 Discussion and Conclusion

KSA plans to use treated wastewater as a major water source and aims to achieve 100% of the required wastewater collection and treatment services for every city with a population above 5,000 by 2025. The Jizan region, KSA is used as case study for considering locations, types, and sizes of sewer pipe and WWTPs. The model treats wastewater (generated at source nodes) at WWTP nodes for minimum costs. The optimization model was developed in GAMS using MINLP. The region is divided into five different zones based on topography limitations, amount of wastewater produced, the importance of candidate locations of WWTPs, and administration boundaries.

This study proposes 8 new WWTPs along with the currently planned or existing 9 WWTPs. It also proposed 20 sewage pipe plans connecting the cities, with a total cost of $ 269.5 million, as shown in Table 6.3. According to KAUST (2011), there are 22 planned WWTPs in the Jizan region, with total generated wastewater of 381,000 m^3/day in 2035. In this study, the total planned WWTPs are 17 and total generated wastewater is 219,000 m^3/day. To date, the government planning system for the region considers only 11 WWTPs for cities that have a large population surrounded by towns or villages. The planning for this study considers over 34 cities with a total population of over 1.3 million people. Each of these cities has a population of at least 5,000 people as of 2017. However, these models present the first step of the planning system, which could be expanded to include hydraulic design, as discussed in Chapter 7. The topography of the region shows that the need to include lift stations might be necessary in the system, which is a major assumption of this study. Finally, the total costs to allocate a sewage network system and WWTPs for the Jizan region, KSA is estimated to be $269.5 million. Table 6.4 shows the summary of numbers, locations, and sizes of WWTPs and total costs in each zone. This study shows that the government-planned WWTPs at Alaliya and Al Shugayri are not necessary, as presented in our planning approach. This study proposed new locations for WWTPs, such as Al Shuqaiq, Al Haqu, Harub, Qawz al Ja'afirah, Al Sehi, Al Qasabah, and Al Aridhah. As described in previous work, the INL method can be used for any collection type system, such as regional wastewater systems. There are two possible locations for WWTPs: centralized systems and decentralized systems. The solution is a trade-off between the transportation costs (sewage network) and WWTP costs. However, the study shows all proposed WWTP are selected, which indicates that costs of allocating WWTP in these cities is cheaper than transporting wastewater to another city for treatment in many cases. Treated wastewater can be used by agriculture, landscaping, and industry in each city. Dummy nodes in the system represent imaginary nodes that are used only for the purposes of connecting nodes to WWTPs (outlets), since the outlets for the system must be on the same INL. The connectivity for each sewage network shows that the model finds the optimum solution by installing WWTPs in each group of cities.

The model is likely to select all possible WWTP locations due to the cost function. For example, in Zone 1, if there is a candidate location of WWTP at Itwide (n32), the optimization model would not choose the candidate location with the lowest cost. In Zone 3, it tends to avoid collected wastewater generated at another city to minimize the volume of flow rate, which leads to massive costs in the sewage system. For instance, the model selected a sewage pipe connecting Al Kadami (n14) to Damad (n5) with a total distance of 17.5 km. Finally, possible locations of WWTPs and pipe connection between cities are found through good engineering judgment and topography limitation. The advantage of these models is that they can be modified for future strategic plans.

Table 6.3. The Summary of Numbers, Locations, Sizes of Sewage Pipe for Each Zone.

Zone	Proposed Sewage Pipe	From	To	Total Length (km)	Design Discharge m3/day
Zone 1	1	n32	n28	19	766
Zone 2	3	n21	n22	18	2,154
		n12	n22	10	1,058
		n23	n9	9.6	2,719
Zone 3	6	n31	n7	9	4,489
		n33	n24	12.7	3,625
		n24	n11	16	1,156
		n14	n11	17.5	2,692
		n11	n5	6	3,040
		n10	n13	17	4,248
Zone 4	2	n25	W-1	6	3,876
		n1	W-1	16	25,673
Zone 5	8	n34	n17	11	2,679
		n27	n26	8.6	1,488
		n20	W-19	6.4	2,754
		n19	W-19	6.3	1,543
		n18	C-1	14.9	1,257
		C-1	n8	15	1,257
		n16	n8	12.9	3,739
		n29	W-4	8.5	5,725

Table 6.4. The Summary of Numbers, Locations, Sizes of WWTP for Each Zone with Total Costs.

Zone	Total number of WWTP	Location of WWTP	WWTP Capacity	Planned WWTP Capacity	Total Costs
			m^3/day	m^3/day	$ million
Zone 1	2	W-6	6,000	5000	$10.5
		W-28	5,000	Proposed by this study	
Zone 2	2	W-22	5,000	Proposed by this study	$14
		W-9	9,000	16,000	
Zone 3	5	W-7	11,000	4,000	$91
		W-30	3,000	Proposed by this study	
		W-5	19,000	16,000	
		W-13	8,000	Proposed by this study	
		W-2	34,000	44,000	
Zone 4	2	W-1	30,000	64,000	$44
		W-15	8,000	Proposed by this study	
Zone 5	6	W-3	30,000	30,000	$110
		W-8	22,000	12,000	
		W-4	25,000	15,000	
		W-17	6,000	Proposed by this study	
		W-26	9,000	Proposed by this study	
		W-19	5,000	Proposed by this study	

7 OPTIMIZATION MODEL OF DESIGN REGIONAL WASTEWATER SYSTEMS: CASE STUDY: JIZAN REGION, KSA

7.1 Introduction

The objectives of this chapter are to present a new methodology for designing regional wastewater systems to include the hydraulic designs of pump stations, commercial pipe diameters and slopes, and capacities of WWTPs, and to apply this model to the Jizan region in KSA. The model developed in this chapter advances the previous model developed in Chapter 6 to include pipe design considering commercial diameters and pipe slopes.

The optimization model, an MINLP problem, was developed using both GAMS and solver for BARON. The layout design model is developed in Chapter 6, which aims to minimize the total cost of sewage networks and WWTPs. The focus is on expansion and consideration of hydraulic components of regional wastewater systems, such as location and size of pump stations, and commercial diameters, pipe slopes, and capacities of WWTP using an optimization model. The main purpose of this model is to introduce pump stations in the system and check whether pumps are needed in strategic planning or not.

7.2 Mathematical Formulation

The objective function minimizes total costs for the sewage network, pump stations, and WWTP while satisfying all hydraulic constraints. Thus, it includes treatment costs as a function of the amount of wastewater that would be treated. The objective function is

$$Min\ Cost = \sum_{i,j}\sum_{p} C_P XP_{i,j,p} X_{i,j} + \sum_{i,j}\sum_{D} \mathrm{C_D}\ \mathrm{L_{j,k,D}} + \sum_{j,k}\sum_{p} C_P XP_{j,k,p} Y_{j,k} +$$

$$\sum_{\mathrm{j,k}}\sum_{\mathrm{D}} \mathrm{C_D}\ \mathrm{L_{j,k,D}} + \sum_{\mathrm{k}} C_{WWTP,k}(\mathrm{QT}_k) \quad (7\text{-}1)$$

Where:

C_P: The cost of pumping head per meter.

$\mathrm{C_D}$: The cost per unit length of diameter for each sewage pipe connecting.

$XP_{i,j}$: Unit head pump added to the system from nodes on INL i to nodes on INL j.

$XP_{j,k}$: Unit head pump add to the system from INL j to WWTP nodes on INL k.

$L_{i,j,D}$: Length pipe of diameter D connecting nodes on INL i to nodes on INL j.

$L_{j,k,D}$: Length pipe of diameter D connecting on INL j to WWTP nodes on INL k.

$X_{i,j}$: 0/1 variable for pumps location from nodes on INL i to nodes on INL j.

$y_{j,k}$: 0/1 variable for pumps location from INL j to WWTP nodes on INL k.

$\mathrm{C_{WWTP,k}}$: The costs of the treated wastewater at nodes on INL k.

$\mathrm{QT_k}$: Flow rate of the treated wastewater discharged from nodes on INL k.

Subject to

a) Hydraulic Constraints

1- Energy equation

The commercial diameter is defined by Manning's equation between nodes on INL i and nodes on INL j to outlet nodes k (WWTP).

$$\left(-\left(SE.us_{i,j}\right) + \left(SE.ds_{i,j}\right)\right) \leq \sum_p XP_{i,j,p} X_{i,j} - \sum_{\mathrm{D}} \left(\frac{\mathrm{m_D}^2 \mathrm{Q_{i,j}^2} \mathrm{n}^2}{\mathrm{D}^{\frac{16}{3}}}\right) \mathrm{L_{i,j,D}} \quad \forall\ (i,j) \quad (7\text{-}2)$$

$$\left(-\left(SE.us_{j,k}\right) + \left(SE.ds_{j,k}\right)\right) \leq \sum_p XP_{j,k,p} Y_{j,k} - \sum_{\mathrm{D}} \left(\frac{\mathrm{m_D}^2 \mathrm{Q_{j,k}^2} \mathrm{n}^2}{\mathrm{D}^{\frac{16}{3}}}\right) \mathrm{L_{j,k,D}} \ \forall\ (j,k) \quad (7\text{-}3)$$

Where:

$Q_{i,j}$: Flow rate carried from nodes on INL i to nodes on INL j.

$Q_{j,k}$: Flow rate carried from nodes on INL j to WWTP nodes on INL k.

$SE.us_{i,j}$: Surface elevation upstream for nodes on INL i to nodes on INL j.

$SE.ds_{i,j}$: Surface elevation downstream for nodes on INL i to nodes on INL j.

$SE.us_{j,k}$: Surface elevation upstream for nodes on INL j to WWTP nodes on INL k.

$SE.ds_{j,k}$: Surface elevation downstream for nodes on INL j to WWTP nodes on INL k.

m_D= 2.16 for U.S. units (3.21 for SI units).

n= Manning's value (0.013)

D= Commercial diameters in (mm).

2- Length constraints

Length constraints force the sum of lengths of commercial diameters to be equal to the total reach length required. Further, more than one diameter for each pipe link can be considered for the pipe link associated between two nodes (i and j) and (j and k), which is of known value.

$$\sum_D L_{i,j,D} = L_{i,j,required} \qquad \forall\ (i,j) \qquad (7\text{-}4)$$

$$\sum_D L_{j,k,D} = L_{j,k,required} \qquad \forall\ (j,k) \qquad (7\text{-}5)$$

Where:

$L_{i,j,required}$: Length required from nodes on INL i to nodes on INL j.

$L_{j,k,required}$: Length required from nodes on INL j to WWTP nodes on INL k.

b) Velocity constraints

The velocity in the pipe must be less than a maximum permissible velocity (to prevent any effects of high velocity flows) and greater than a minimum permissible velocity (to prevent deposition). The velocity constraints are defined as

$$V_{max} \geq V_{(i,j)} \geq V_{min} \qquad \forall\ (i,j) \tag{7-6}$$

$$V_{max} \geq V_{(j,k)} \geq V_{min} \qquad \forall\ (j,k) \tag{7-7}$$

Velocity constraints are included in the objective function using a penalty method, as defined below

$$V_{(i,j)} = \begin{cases} V_{(i,j)}\mathrm{BigM} & , V_{(i,j)} < V_{min} \\ V_{(i,j)} & , V_{(i,j)} \geq V_{min} \end{cases} \tag{7-8}$$

$$V_{(i,j)} = \begin{cases} V_{(i,j)} & , V_{(i,j)} < V_{max} \\ V_{(i,j)}\mathrm{BigM} & , V_{(i,j)} \geq V_{max} \end{cases} \tag{7-9}$$

Where BigM is a penalty value = 10^{10}.

The minimum and maximum velocities are assumed to be 0.6 m/sec and 2.6 m/sec (SI units), respectively.

7.3 Model Application

The Jizan Region was described in detail in previous chapters. The sewer pipes are connected between each city and all lead to the WWTP. This is explained in the previous chapter, where the total number of WWTPs was around 17 and a total placement was around 20 sewage pipe connections between cities. As described in previous chapters, the case study is divided into five different zones and the layout of the sewer pipes and WWTP is identified for each zone. Table 7.1 shows the proposed sewage network for the Jizan region from upstream elevation to downstream elevation, including cities, WWTP, elevations, and distances.

For cost functions, the candidate commercial diameters costs are defined as a function of unit length (m). This includes pipe price, excavation work, and replacement costs. The commercial diameters are in millimeters (mm) and costs are in dollars, as shown in Table 7.2. The pump costs range from 4 million SAR to 40 million SAR (for 100 kW to 2000 kW). Ultimately, this adds up to $15,000 for each unit head in meters. This information is data derived from MEWA and previous projects in KSA.

Table 7.1. The Proposed Sewage Network for Jizan Region.

Zones	Proposed sewage			Distance			
	Upstream			Downstream			
	City	Node number	Elevation (m)	City/ WWTP	Node number	Elevation (m)	Km
Zone 1	Itwide	n32	30	Al Shuqaiq	n28	10	19
Zone 2	Al Reeth	n21	800	Al Haqu	n22	200	18
	Al Fatiha	n12	220	Al Haqu	n22	200	10
	Masliyah	n23	400	Baish	n9	200	9.6
Zone 3	Fayfa	n31	1200	Addayer	n7	800	9
	Aiban/Belghazi	n33	400	Alaydabi	n24	230	12.7
	Alaydabi	n24	230	Al Shugayri	n11	110	16
	Al Kadami	n14	100	Damad	n5	80	17.5
	Al Shugayri	n11	110	Damad	n5	80	5
	Alaliya	n10	50	Qawz al Ja'afirah	n13	30	17
Zone 4	Al Madaya	n25	10	WWTP	W-1	0	6
	Jizan	n1	0	WWTP	W-1	0	16
Zone 5	Al Mwassam	n34	15	Al Sehi	n17	0	11
	Alhumira	n27	230	Al Aridhah	n26	200	8.6
	Al Qasabah	n20	250	WWTP	W-19	200	6.4
	Khushal	n19	250	WWTP	W-19	200	6.4
	AlKhubah/Alharth	n18	250	Collection node	C-1	180	14.9
	Collection node	C-1	180	Ahad Al Masarihah	n8	80	15
	AlGofol	n16	120	Ahad Al Masarihah	n8	80	12.9
	Al Tuwal	n29	70	WWTP	W-4	70	8.5

Table 7.2. Pipe Costs per Unit Length.

Pipe costs	
mm	$/m
200	60
250	80
300	99
375	130
450	165
525	200
600	235
675	260
750	310
900	390

7.3.1 Zone 1

In Zone 1, the model is developed to design the sewer pipe from Itwide (n32) to Al Shuqaiq (n28), with a total distance of 19 km. Itwide is small city and has a population of around 5,000 and generates 766 m^3/day of wastewater. The layout design shows the volume of generated wastewater that would transfer to Al Shuqaiq (n28), with a total design capacity of 5,000 m^3/day. The difference in elevation is 20 m, along with a total distance of 19 km. Figure 7.1 shows the hypothetical layout design of the system. Green represents agricultural areas and purple represents residential boundaries. The figure also shows locations of cities and proposed WWTPs with total capacities.

Figure 7.1. Layout Design of the System for Zone 1.

7.3.1.1 Results

The model results from Zone 1 show that that the diameter is 200 mm and the pump station is 50 m, satisfying the velocity constraint in the pipe. The interesting point is that the generated wastewater flow rate at Itwid is low (766 m^3/day), which must be pressurized to maintain velocity constraints. The total costs for Zone 1 are $11.9 million, which is higher than layout design model, which is $10.5 million.

7.3.2 Zone 2

In Zone 2, the model was developed to design three sewer pipes:

1) Al Reeth (n21) to Al Haqu (n22), with a total distance of 18 km and generated wastewater of 2,154 m^3/day.
2) Al Fatiha (n12) to Al Haqu (n22), with a total distance of 10 km and generated wastewater of 1,058 m^3/day.

3) Masliyah (n23) to Baish (n9), with a total distance of 9.6 km and generated wastewater of 2,719 m^3/day.

The generated wastewater flow rates at Al Reeth, Al Fatiha, and Masliyah are 2,154 m^3/day, 1,058 m^3/day, and 2,719 m^3/day, respectively. The capacities for the proposed WWTPs are 5,000 m^3/day and 9,000 m^3/day at Al Haqu and Baish, respectively. Figure 7.2 shows the hypothetical layout design of the system. Green represents agricultural areas and purple represents residential boundaries. The figure also shows locations of cities and proposed WWTPs with total capacities.

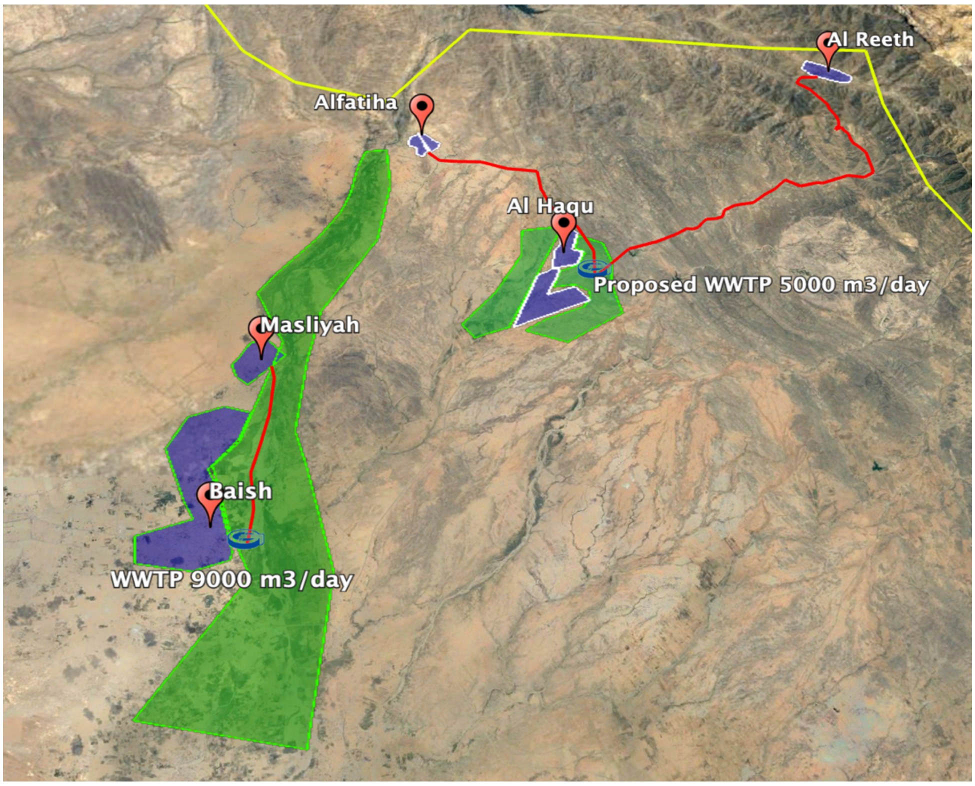


Figure 7.2. Layout Design of the System for Zone 2.

7.3.2.1 Results

In Zone 2, the results show that the total costs are \$15.6 million, which are higher than the total costs of the \$14 million layout design. The commercial diameters are only 200 mm due to small values of generated wastewater. For the sewer pipe connecting Masliyah (n23) to Baish (n9), there are two sections: the first one has a commercial diameter of 200 mm for 3.6 km and the second one has a commercial diameter of 250 mm for 6 km. The pump station is needed for the sewer pipe connecting Al Fatiha to Al Haqu as the difference in elevation is 20 m and the total length is 10 km, which does not satisfy velocity constraints.

7.4.3 Zone 3

In Zone 3, the model is developed to design six sewer pipes:

1) Fayfa (n31) to Addayer (n7), with a total distance of 9 km and generated wastewater of 4,489 m^3/day.
2) Aiban/Belghazi (n33) to Alaydabi (n24), with a total distance of 12.7 km and generated wastewater of 3,625 m^3/day.
3) Alaydabi (n24) to Al Shugayri (n11), with a total distance of 16 km and generated wastewater of 4,781 m^3/day.
4) Al Kadami (n14) to Damad (n5), with a total distance of 17.5 km and generated wastewater of 2,692 m^3/day.
5) Al Shugayri (n11) to Damad (n5), with a total distance of 6 km and generated wastewater of 7,820 m^3/day.
6) Alaliya (n10) to Qawz al Ja'afirah (n13), with a total distance of 17 km and generated wastewater of 4,248 m^3/day.

The WWTPs located at Harub, Qawz al Ja'afirah, Damad, Addayer, and Sabya have total capacities of 3,000 m^3/day, 8,000 m^3/day, 20,000 m^3/day, 11,000 m^3/day, and 34,000 m^3/day, respectively. Figure 7.3 shows the hypothetical layout design of the system. Green represents agricultural areas and purple represents residential boundaries. The figure also shows locations of cities and proposed WWTPs with total capacities.

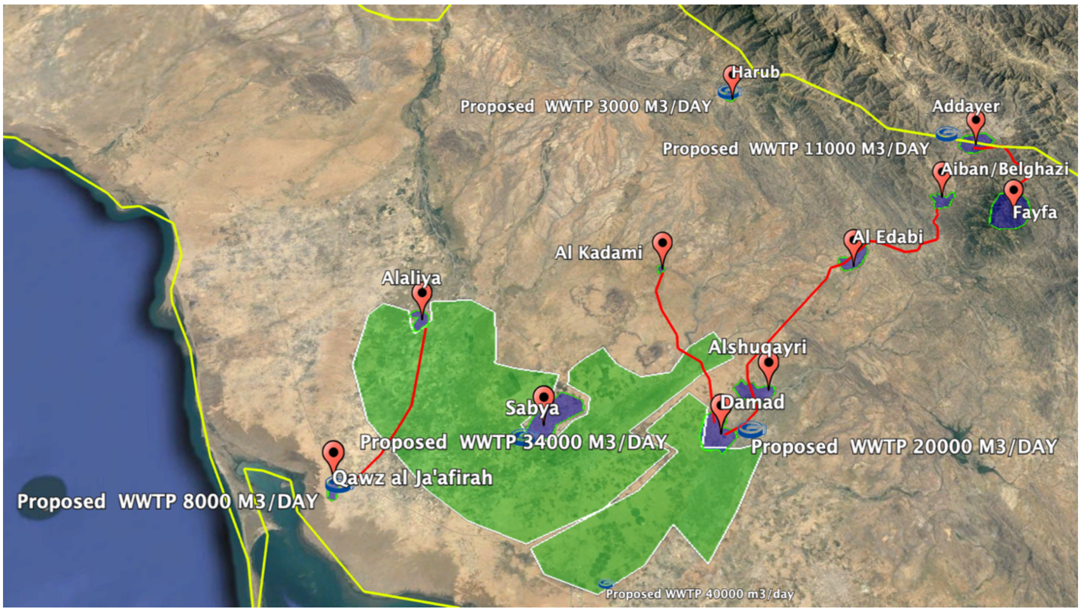


Figure 7.3. Layout Design of the System for Zone 3.

7.4.3.1 Results

In Zone 3, the results show that the total costs are an estimated \$92 million, slightly higher (1%) than the layout design. The commercial diameters range from 300 mm to 525 mm. The pump station is needed for Al Kadami (52 m) and Al Shugayri (33 m), since the difference in elevations are 20 m and 30 m, with a total distance of 17.5 km and 5 km, respectively. Even though the difference in elevation between Al Shugayri and Damad is around 30 m, the model requires a 33 m pump head to satisfy the velocity requirement. Furthermore, the commercial diameters from Al Shugayri (n11) to Damad (n5) are 2.7 km for 450 mm and 3.2 km for 525 mm, respectively. The increase in commercial diameters is due to the increase of generated wastewater to 7,820 m^3/day.

7.3.4 Zone 4

In Zone 4, the model is developed to design two sewer pipes:

1) Al Madaya (n25) to existing WWTP (W-1), with a total distance of 6 km and generated wastewater of 25,673 m^3/day.
2) Jizan (n1) to (W-1), with a total distance of 16 km and generated wastewater of 3,876 m^3/day.

The difference in elevation for first sewer pipe is 10 m, while the difference in elevation for the second sewer pipe is zero. The proposed capacities of WWTPs are 8,000 m^3/day in Wadi Jizan and 30,000 m^3/day south of the city of Jizan. Figure 7.4 shows the hypothetical layout design of the system. Green represents agricultural areas and purple represents residential boundaries. The figure also shows locations of cities and proposed WWTPs with total capacities.

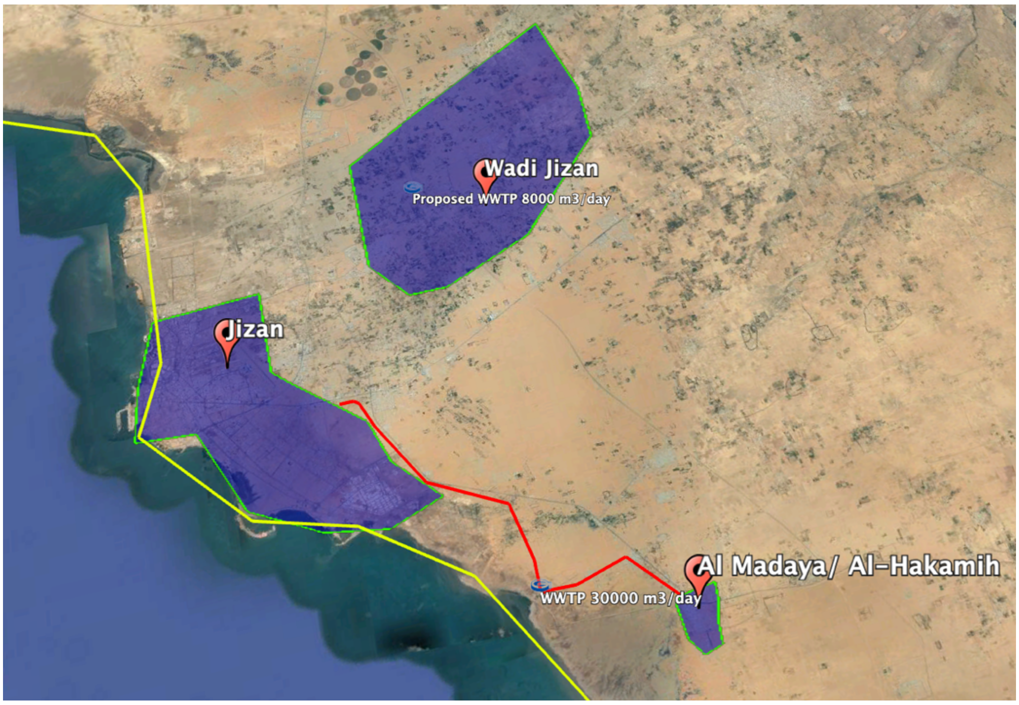


Figure 7.4. Layout Design of the System for Zone 4.

7.3.4.1 Results

In Zone 4, the commercial diameters are determined to be 675 mm from Jizan (n1), due to the highest potential flow rates of 25,673 m^3/day. Pump stations are needed for both cities, since the difference in elevations is almost zero. The total costs are estimated to be $49 million, which is 10% higher than the layout design. This is due to the fact that the highest costs are associated with pump stations and commercial diameters.

7.3.5 Zone 5

In Zone 5, the sewer pipe connects the following:

1) Al Mwassam (n34) to Al Sehi (n17), with a total distance of 11 km and generated wastewater of 2,679 m^3/day.
2) Alhumira (n27) to Al Aridhah (n26), with a total distance of 8.6 km and generated wastewater of 1,488 m^3/day.
3) Al Qasabah (n20) to W-19, with a total distance of 6.4 km and generated wastewater of 2,754 m^3/day.
4) Khushal to W-19, with a total distance of 6.4 km and generated wastewater of 1,543 m^3/day.
5) Al Khubah/Alharth (n18) to collection point (C-1), with a total distance of 14.9 km and generated wastewater of 1,257 m^3/day.
6) Collection point to Ahad Al Masarihah (n8), with a total distance of 15 km and generated wastewater of 1,257 m^3/day.
7) Al Gofol (n16) to Ahad Al Masarihah (n8), with a total distance of 12.9 km and generated wastewater of 3,739 m^3/day.

8) Al Tuwal (n29) to WWTP (W-4), with a total distance of 8.5 km and generated wastewater of 5,725 m^3/day.

The proposed WWTPs are located at Al Sehi, Al Aridhah, Khushal, Abu Arish, Samtah, and Ahad Al Masarihah, with total capacities of 6,000 m^3/day, 9,000 m^3/day, 5,000 m^3/day, 30,000 m^3/day, 25,000 m^3/day, and 22,000 m^3/day, respectively. Figure 7.5 shows the hypothetical layout design of the system. Green represents agricultural areas and purple represents residential boundaries. The figure also shows locations of cities and proposed WWTPs with total capacities.

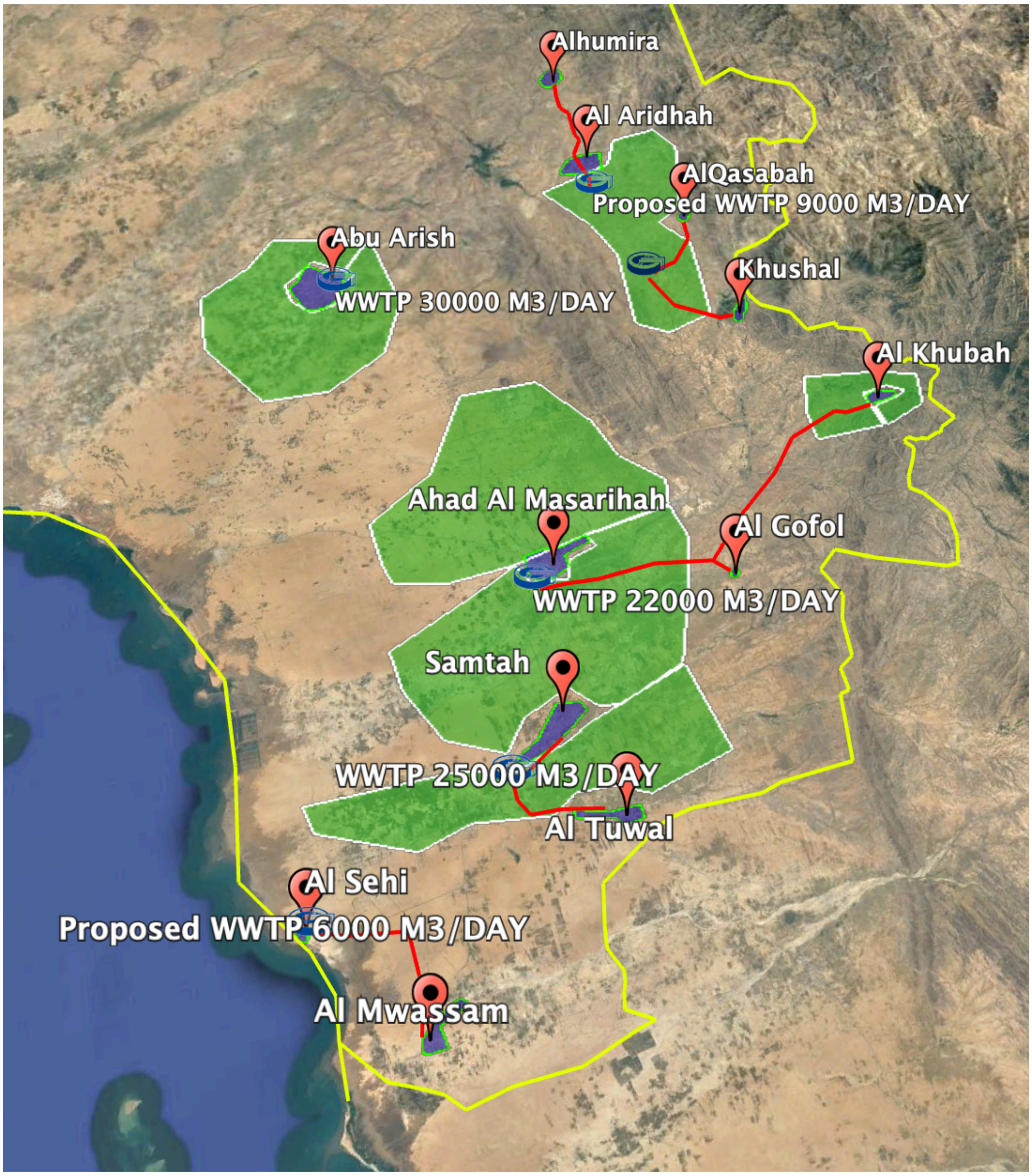


Figure 7.5. Layout Design of the System for Zone 5.

7.3.5.1 Results

In Zone 5, which is considered to be the largest zone and has the highest total cost (around $111 million), the model is developed for eight sewer pipes. Pump stations are needed at two locations: Al Tuwal and Al Mwassam. The commercial diameters range from 200 mm to 450 mm. The pump station at Al Tuwal is 19 m and the commercial diameter is 450 mm due to generated wastewater of 5,725 m^3/day, with zero difference in elevation. Most sewage pipes have been divided into two commercial diameters, reducing the total costs in comparison to having only one commercial diameter.

7.4 Discussion and Conclusions

The optimization model was formulated using MINLP and solved in GAMS. This model is applied to the case study, the Jizan region, KSA. The region is divided into five zones and the layout of the sewer network and locations of WWTP for each zone were defined in Chapter 6. This study considers hydraulic components of regional wastewater systems, such as location and size of pump stations, commercial diameters, slopes, and capacities of WWTPs using the optimization model.

By observation, the need for pump stations is increased when the differences in elevations are small and the distance is greater. The cost function of pump stations has a significant impact on whether to allocate a pump station or not. For instance, if the pump cost is low, the optimization model must select a large pump head in order to select small commercial diameters and to satisfy the velocity in the sewage pipe. When the pump cost is large, the optimization model selects extensive excavation work and commercial diameters to avoid higher costs. Thus, there are trade-offs between pump costs and excavation work costs, which can be solved by optimization models. The study shows the importance of cost functions and their effect on the layout and hydraulic design of the systems. The excavation work costs are included in the cost of the commercial diameters per unit length. Excavation costs are influenced by many factors, such as crown elevations, slopes, and commercial diameters. The purpose of M-2 is to check whether pump stations are necessary in strategic planning of the region. The optimization models select the minimum costs associated with sewage pipe and WWTP design. The total costs of M-2 are 3.8% higher than the layout design M-1 ($279.8-$269.5 million). The large total costs include the installation, operation, and maintenance of WWTPs. In Zone 4, M-2 incurs a total cost of around $5 million more than M-1 due to the hydraulic design components and cost functions associated with it. The increase in cost also comes from pump stations and commercial diameter costs in the system. The results of the designed wastewater system for the Jizan region are shown in Table 7.3.

Table 7.3. The Summary of Hydraulic Design for Regional Wastewater System, Jizan Region, KSA.

Zones	Locations				Length		Diameter		Pump Station	Total Costs in Millions
	Upstream		Downstream		L		D		Pump Head	
	Node	m	node	m	km	km	mm	mm	m	
Zone 1	n32	30	n28	10	19	------	200	------	50	$11.9
Zone 2	n21	800	n22	200	18	------	200	------	0	$15.6
	n12	220	n22	200	10	------	200	------	39	
	n23	400	n9	200	3.6	6	200	250	0	
Zone 3	n31	1200	n7	800	9	------	300	------	0	$91.6
	n33	400	n24	230	12.7	------	300	------	0	
	n24	230	n11	110	16	------	300	------	0	
	n14	100	n5	80	17.5	------	300	------	52	
	n11	110	n5	80	2.8	3.2	450	525	33	
	n10	50	n13	30	17	------	375	------	0	
Zone 4	n25	10	W-1	0	6	------	375	------	6	$49
	n1	0	W-1	0	16	------	675	------	72	
Zone 5	n34	15	n17	0	11	------	300	------	30	$111.3
	n27	230	n26	200	8.6	------	250	------	0	
	n20	250	W-19	200	3	3.4	250	300	0	
	n19	250	W-19	200	3.1	3.2	200	250	0	
	n18	250	C-1	180	12.6	2.3	300	375	0	
	C-1	180	n8	80	2.5	12.5	250	300	0	
	n16	120	n8	80	5.1	7.8	250	300	0	
	n29	70	W-4	70	8.5	------	450	------	19	

7.5 Notation

Sets

i: Set of sources nodes on INL i.

j: Set of collection and/or sources nodes on INL j.

k: Set of (outlet) WWTP nodes on INL k.

D: Set of possible pipe diameters (200 mm, 250 mm, 300 mm, 375 mm, …. 900 mm).

P: Set of pump stations in the system.

Parameters

$Q_{i,j}$: Flow rate carried from nodes on INL i to nodes on INL j.

$Q_{j,k}$: Flow rate carried from nodes on INL j to WWTP nodes on INL k.

$L_{i,j,required}$: Length required from nodes on INL i to nodes on INL j.

$L_{j,k,required}$: Length required from nodes on INL j to WWTP nodes on INL k.

$SE.us_{i,j}$: Surface elevation upstream for nodes on INL i to nodes on INL j.

$SE.ds_{i,j}$: Surface elevation downstream for nodes on INL i to nodes on INL j.

$SE.us_{j,k}$: Surface elevation upstream for nodes on INL j to WWTP nodes on INL k.

$SE.ds_{j,k}$: Surface elevation downstream for nodes on INL j to WWTP nodes on INL k.

C_D: The cost per unit length of diameter for each connecting sewage pipe.

C_P: The cost of pump heads per meter.

$C_{WWTP,k}$: The costs of the treated wastewater at nodes on INL k.

QT_k: Flow rate of the treated wastewater discharged from nodes on INL k.

V_{max}, V_{min}: Min-max allowable velocity in the pipe (0.6 m/sec and 2.6 m/sec).

m_D= 2.16 for U.S. units (3.21 for SI units).

n= Manning's value (0.013)

d= Pipeline diameter in mm.

Variables

$XP_{i,j}$: Unit head pump added to the system from nodes on INL i to nodes on INL j.

$XP_{j,k}$: Unit head pump added to the system from INL j to WWTP nodes on INL k.

$L_{i,j,D}$: Length of pipe of diameter D connecting nodes on INL i to nodes on INL j.

$L_{j,k,D}$: Length of pipe of diameter D connecting on INL j to WWTP nodes on INL k.

$X_{i,j}$: 0/1 variable for pumps location from nodes on INL i to nodes on INL j.

$y_{j,k}$: 0/1 variable for pumps location from INL j to WWTP nodes on INL k.

8 OPTIMIZATION MODEL FOR LAYOUT AND PIPE DESIGN FOR STORM WATER SYSTEMS

8.1 Introduction

The objectives of this chapter are 1) to present a new methodology for the optimal layout and pipe design for storm water systems, 2) to explain the development of the mathematical formulation of MINLP using optimization model in GAMS, and 3) to apply the model using four different scenarios with varying parameters, such as pipe length, ground surface elevations, and design discharges.

8.2 Preparation for Model Formulation

The INL method for a storm water system is explained in detail by Mays et al. (1976) and Steele et al. (2016). The same principle for solving the layout design problem is used in this chapter. However, there are many challenges to including pipe design in the mathematical formulation, such as the introduction of commercial diameters, velocity, and/or slope constraints, and crown elevation constraints. Commercial diameters are considered to be a function of unit pipe length. The first approach considers one or more commercial diameters in selecting pipe, while the second approach considers only one commercial diameter in selecting pipe. The difference between the two approaches is observed in the objective function and velocity constraints. The advantages of using these approaches are providing commercial diameters for sewer pipes and using cost functions for commercial diameters as a function of pipe length, which includes equipment and installation costs associated with commercial diameters. On the other hand, construction costs for manholes are another cost factor that should be implemented in a mathematical formulation that depends on cutting depth from the ground surface. Generally, there is a trade-off between the costs of commercial diameters and manhole construction. Larger excavation depths lead to larger manhole construction costs. The maximum velocity constraint forces the optimization model to choose steeper slopes for smaller diameters, which in turn leads to a manhole with a deeper hole.

The aim of the optimization model presented in this chapter is to provide an optimal storm water sewer network design (commercial diameters and excavation depths) that conveys storm water at the required capacity. The decision variables considered for the sewer design optimization model include commercial diameters for sewer pipes and excavation depths (the difference between surface elevation and pipe crown elevations). The constraints for the optimization include Manning's equation for velocity computation and upper and lower bounds on the velocity of water in pipes.

8.2.1 Modifications to Manning's Equation

Manning's equation is widely used to design storm water systems hydraulically. The flow in a storm water sewer network is expressed by Manning' s equation **as follows:**

$$\mathrm{E2} - \mathrm{E1} = \left(\frac{\mathrm{m_D}^2 \mathrm{Q}^2 \mathrm{n}^2}{\mathrm{D}^{\frac{16}{3}}}\right) \mathrm{L} \qquad (8\text{-}1)$$

Where $\mathrm{m_D}$ is 2.16 for U.S. units (3.21 for SI units). Q is flow rate (gpm, Mgal/d, or cfs in U.S. or m3/s, ML/d, or L/s in SI units) and D is rounded up to the next commercial size pipe (pipe diameter in meters in SI units or inches in U.S. units). Then, n is Manning's value, which depends on the pipeline material. $\mathrm{S_0}$ is the slope of hydraulic grade line, dimensionless $\mathrm{S} = \mathrm{E2} - \mathrm{E1/L}$, where $\mathrm{E1}$ is surface water elevation upstream, E2 is surface water elevation downstream in feet or meters, and L is pipe length in feet or meters. As in previous chapters, flow is assumed to be steady and uniform.

8.2.2 Minimum and Maximum Velocity in the System

For a gravity flow system, the velocity has to be between a maximum permissible velocity (to prevent pipe abrasion) and a minimum permissible velocity (to prevent deposition) (Mays, 2001). The candidate commercial diameters are defined as parameters in this model. In the first approach, the authors used the concepts of minimum and maximum slopes as an alternative to velocity constraints to avoid unfeasibility of some candidate commercial diameters. The best way is provided by Hsie et al. (2019), who defined the problem in terms of minimum and maximum slope, using Manning's equation (8-1).

Table 8.1 shows the minimum and maximum slopes for each commercial diameter. Upper and lower bounds for sewer pipe slopes of 0.5% and 2%, respectively, were imposed on commercial diameters of 8" to 30". However, these values can be adjusted for different commercial diameters based on the type and size of the problem.

Table 8.1. Minimum and Maximum Slope for Different Commercial Diameter.

Diameters	**Md**	**n**	**Area**	**V (ft/sec)**		**Slope**	
In			ft2	Min	Max	Min	Max
8	2.16	0.013	0.349	2.5	12	0.52%	12.01%
10	2.16	0.013	0.545	2.5	12	0.39%	8.92%
12	2.16	0.013	0.785	2.5	12	0.30%	7.00%
15	2.16	0.013	1.227	2.5	12	0.23%	5.20%
18	2.16	0.013	1.766	2.5	12	0.18%	4.07%
21	2.16	0.013	2.404	2.5	12	0.14%	3.32%
24	2.16	0.013	3.140	2.5	12	0.12%	2.78%
30	2.16	0.013	4.906	2.5	12	0.09%	2.06%

For the second approach, a binary variable (1 for selection of a particular pipe diameter or 0 otherwise) was introduced to select one commercial diameter. A velocity equation is formulated for representing the binary variable as follows:

$$V_{min} \sum_{\mathrm{D}} \text{Binary Variable}_{\mathrm{D}} \leq \mathrm{V} \leq V_{max} \sum_{\mathrm{D}} \text{Binary Variable}_{\mathrm{D}}$$

If the binary variable selected one commercial diameter, the velocity of the pipe has to be between two values, otherwise the velocity is zero for that commercial diameter.

8.2.3 Cost Functions

The objective of the optimization model developed herein is to minimize the total cost, which is based on cost functions for commercial diameter, pipe length, excavation, and manhole construction. For cost data, the total cost for pipeline per unit length is presented in Table 8.2. It is assumed that cost values include pipe price, pavement removal for excavation, trench excavation, and final backfill; the price for permanent pavement replacement is also added to total costs for each commercial diameter by using \$40 per square yard, which is \$4.45 per square feet. The pavement replacement depends on the width of the trench, which is considered to be 3 ft plus commercial diameter size. On the other hand, manhole costs are highly dependent on cutting depth from surface elevations, which are developed in equation (8-2). These values were used by Karovic and Mays (2014) to design sewer storm systems.

Cost(\$/depth) = 1818.2 (Cutting Depth @ upstream-ft) -1000. (8-2)

Table 8.2. Total Costs of Commercial Diameters per Unit Foot

Pipe Cost			
(in)	(1) ($/ft)	(2) Pavement Replacement ($/ft)	Final Total Costs ($/ft)
8	18	16	34
10	24	17	41
12	30	17	47
15	39	18	57
18	50	20	70
21	60	21	81
24	80	22	102
30	90	24	114

8.3 The Mathematical Formulation

The mathematical formula consists of an objective function, a set of constraints, and decision variables. Two mathematical formulas for this chapter aim to simultaneously find the layout and pipe design for a storm water network system. The method uses the INL method, where the water would flow from the upstream manhole on the INL n to the downstream manhole on the INL n+1.

8.3.1 Objective Function

The objective function minimizes the total costs for layout and pipe design for all system elements associated with commercial diameters, slopes, and crown elevations simultaneously. Two objective functions considered for the aforementioned two approaches are expressed as follows:

$$\text{Min } \sum_{n}\sum_{m_{n+1}}\sum_{m_n}\left(\left(\sum_{D} C_D\, L_{n,m_n,m_{n+1},D}\right) + C_{n,m_n}\, CE.us_{n,m_n,m_{n+1}}\right)X_{n,m_n,m_{n+1}} \quad (8\text{-}3a)$$

Where:

C_D: The cost of commercial diameter per unit length.

C_{n,m_n}: The cost of manhole m_n associated with cutting depth to upstream crown elevation.

$L_{n,m_n,\, m_{n+1},D}$: Length of pipe of the commercial diameter D connecting manhole m_n on INL n to manhole m_{n+1} on INL n+1.

$C\,E.\,us_{n,m_n,m_{n+1}}$: Cutting depth to upstream crown elevation for manhole m_n on INL n to manhole m_{n+1} on INL n+1.

$X_{n,m_n,m_{n+1}}$: 0-1 binary variable where 1 indicates a pipe connection between manholes m_n and m_{n+1}.

Equation (8-3a) was applied to the first approach, which considers the minimum one or more commercial diameters per unit length and cutting depth at upstream manholes.

$$\text{Min } \sum_n \sum_{m_{n+1}} \sum_{m_n} \Big(\big(\sum_D C_D\, L_{n,m_n,\, m_{n+1}}\, CanD_{n,m_n,m_{n+1},D} \big) + C_{n,m_n}\, CE.\,us_{n,m_n,m_{n+1}} X_{n,m_n,m_{n+1}} \Big) \tag{8-3b}$$

Where:

$CanD_{n,m_n,m_{n+1},D}$: 0-1 binary variable where 1 indicates only one commercial diameter, which is a function of $X_{n,m_n,m_{n+1}}$ (Eq. 13 below) for pipe connection between manholes m_n and m_{n+1}.

Equation (8-3b) was applied to the second approach, which considers only the minimum one commercial diameter per unit length and cutting depth at upstream manholes. The basic different between two approaches is that the first approach has more flexibility to assign one or more commercial diameters for each pipe, while the second approach has only one commercial diameter for each pipe.

8.3.2 Subject To

8.3.2.1 Layout Constraints

A. Continuity constraint:

To define total flows for each manhole m_n on INL n.

$$\sum_{m_{n+1}} Q_{n,m_n,m_{n+1},} X_{n,m_n,m_{n+1}} = \sum_{m_{n-1}} Q_{n,m_{n-1},m_n,} X_{n,m_{n-1},m_n} + QIN_{n,m_n} \ \forall\, n\,, \forall\, m_n \qquad (8\text{-}4)$$

QIN_{n,m_n}: The surface inflow into the manhole m_n on INL n.

$Q_{n,m_n,m_{n+1},}$: Flow through the pipes between manhole m_n on INL n to manhole m_{n+1} to INL n+1.

$Q_{n,m_{n-1},m_n,}$: Flow through the pipes between m_{n-1} on INL n-1 to manhole m_n to INL n.

B. Connectivity constraint

This constraint allows only one drainpipe from manhole m_n to m_{n+1} if there is allowable flow through the pipe.

$$\sum_{m_{n+1}} X_{n,m_n,m_{n+1}} = 1 \text{ if } \sum_{m_{n+1}} Q_{n,m_n,m_{n+1},} > 0 \ \forall\, n\,, \forall\, m_n \qquad (8\text{-}5)$$

8.3.2.2 Pipe Design Constraints

C. Commercial diameters constraints

1) Length constraints

The length constraint is a summation of the lengths of commercial diameters and is equal to the total length required for manhole m_n on INL n to manhole m_{n+1} on INL n+1.

$$\sum_{D} L_{n,m_n,\,m_{n+1},D} = L_{\,n,m_n,\,m_{n+1}\text{required}} X_{n,m_n,m_{n+1}} \ \forall\, n\,, \forall\, m_n, \forall\, m_{n+1} \qquad (8\text{-}6)$$

$L_{n,m_n,\,m_{n+1},\text{required}}$: Required length pipe connecting manhole m_n on INL n to manhole m_{n+1} on INL n+1.

Constraint (8-6) is for one or more commercial diameters in one pipe connection between two INLs (n to n+1). It is only applied to the first approach since the authors introduced the length of certain commercial diameter as a decision variable.

2) Manning's equation constraints

The commercial diameter is defined by Manning's equation for full pipe flow (design flow).

$$\sum_{D} \frac{m_D{}^2 Q^2_{n,m_n,\, m_{n+1}} n^2}{D^{\frac{16}{3}}} L_{n,m_n,\, m_{n+1},D}\, X_{n,m_n,m_{n+1}}$$

$$= ((SE.\,us_{n,m_n,m_{n+1}} - CE.\,us_{n,m_n,m_{n+1}}) - (SE.\,ds_{n,m_n,m_{n+1}} - CE.\,ds_{n,m_n,m_{n+1}}))\, X_{n,m_n,m_{n+1}}$$

$\forall$ n , $\forall\, m_n$, $\forall\, m_{n+1}$ (8-7a)

Where:

$SE.\,us_{n,m_n,m_{n+1}}$: Surface elevation upstream for manhole m_n on INL n to manhole m_{n+1} to INL n+1.

$SE.\,ds_{n,m_n,m_{n+1}}$: Surface elevation downstream for manhole m_n on INL n to manhole m_{n+1} on INL n+1.

$C\,E.\,us_{n,m_n,m_{n+1}}$: Cutting depth to upstream crown elevation for manhole m_n on INL n to manhole m_{n+1} on INL n+1.

$C\,E.\,ds_{n,m_n,m_{n+1}}$: Cutting depth to downstream crown elevation for manhole m_n on INL n to manhole m_{n+1} on INL n+1.

Equation (8-7a) was applied to the first approach, which considers the length of certain commercial diameters into Manning's equation.

$$\sum_{D} \frac{m_D{}^2 Q^2_{n,m_n,\, m_{n+1}} n^2}{D_{n,m_n,m_{n+1}}{}^{\frac{16}{3}}} L_{n,m_n,\, m_{n+1},required}\, X_{n,m_n,m_{n+1}}\, CanD_{n,m_n,m_{n+1},D}$$

$$= ((SE.\, us_{n,m_n,m_{n+1}} - CE.\, us_{n,m_n,m_{n+1}}) - (SE.\, ds_{n,m_n,m_{n+1}} - CE.\, ds_{n,m_n,m_{n+1}}))\, X_{n,m_n,m_{n+1}}$$

$\forall$ n , $\forall$ m_n, $\forall$ m_{n+1} (8-7b)

Equation (8-7b) was applied to the second approach, which considers the binary variable of selection of only one commercial diameter into Manning's equation. The difference between (Eq. 8-7a) and (Eq. 8-7b) is the ability of Manning's equation to consider one or more than one commercial diameters.

D. Minimum and maximum velocity constraints.

This constraint is developed by using maximum and minimum slopes for manhole m_n on the INL n to manhole m_{n+1} on the INL n+1. This constraint is used for the first approach.

$Slope_{n,m_n,m_{n+1}} \leq$ SloMax+BigM (1-$X_{n,m_n,m_{n+1}}$) $\forall$ n, $\forall$ m_n , $\forall$ m_{n+1} (8-8)

SloMin $\leq$ $Slope_{n,m_n,m_{n+1}}$+BigM (1-$X_{n,m_n,m_{n+1}}$) $\forall$ n, $\forall$ m_n , $\forall$ m_{n+1} (8-9)

Where;

$$Slope_{n,m_n,m_{n+1}} = \frac{(SE.us_{n,m_n,m_{n+1}} - E.us_{n,m_n,m_{n+1}})\text{-}(SE.ds_{n,m_n,m_{n+1}} - E.ds_{n,m_n,m_{n+1}})}{L_{\,n,m_n,\, m_{n+1} required}} \quad (8\text{-}10\text{-}a)$$

$Slope_{n,m_n,m_{n+1}}$: Slope of pipes between m_n on INL n to manhole m_{n+1} and INL n+1.

SloMax: Maximum slope is 2% for all candidate commercial diameters (8" to 30").

SloMin: Minimum slope is 0.5% for candidate commercial diameters (8" to 30").

BigM: Plenty value = 10^{10}

Equations (8-8) and (8-9) were applied to the first approach only. Since the authors considered one and/or more than one commercial diameters in the pipe system, the approach of using minimum and maximum slopes is a good way to describe velocities in the system.

$V_{n,m_n,m_{n+1}} \leq \text{ValMax} * \sum_D \text{CanD}_{n,m_n,m_{n+1},D}\ \forall\ n\ ,\forall\ m_n\ ,\forall\ m_{n+1}$ (8-10)

$V_{n,m_n,m_{n+1}} \geq \text{ValMin} * \sum_D \text{CanD}_{n,m_n,m_{n+1},D}\ \forall\ n\ ,\forall\ m_n\ ,\forall\ m_{n+1}$ (8-11)

Where discharge is a function of velocity and area of the pipe which is described in equation Q =VA.

Constraint for selection of a single commercial diameter is provided in (Eq. 8-12):

$\sum_D CanD_{n,m_n,m_{n+1},D} = X_{n,m_n,m_{n+1}}\quad \forall\ n, \forall\ m_n\ , \forall\ m_{n+1}$ (8-12)

Equations (8-10), (8-11), and (8-12) were applied to the second approach only. These equations allow only one commercial diameter for any pipe connecting manhole m_n on the INL n to manhole m_{n+1} on the INL n+1.

E. Continues slope constraints

1) Continues slope constraints to ensure that slope continues in the downstream direction.

$\text{CE.ds}_{n,m_n,m_{n+1}} X_{n,m_n,m_{n+1}} \geq \text{CE.us}_{n,m_n,m_{n+1}} X_{n,m_n,m_{n+1}}\ \forall\ n\ , \forall\ m_n\ , \forall\ m_{n+1}$ (8-13)

2) Junction constraints where the cutting depth upstream of manholes m_{n+1} on the INL n+1 have to be greater or equal to the cutting depth downstream of manholes m_n on the INL n.

$CE.us_{n,m_n,m_{n+1}} \geq CE.ds_{n,m_{n-1},m_n} X_{n,m_n,m_{n+1}}\ \forall\ n\ , \forall\ m_{n-1}, \forall\ m_n, \forall\ m_{n+1}$ (8-14)

F. Minimum pipe cover depth constraints.

1) The cutting ground depth should take into account the minimum cover depth of ground surface elevation to protect the pipe from the damage of heavy traffic loads. Minimum cover depth for scenarios is considered to be 3 ft.

$CE.us_{n,m_n,m_{n+1}} \geq Min_{coverdepth} X_{n,m_n,m_{n+1}}\ \forall\ n\ , \forall\ m_n, \forall\ m_{n+1}$ (8-15)

G. Final downstream pipe elevation constraint

1) Tie-ins to existing sewer systems must also be defined if the sewer being designed connects to outlet sewer. Therefore, the new crown elevation must be lower than the crown elevation of the existing pipe ET.

$\mathrm{SE.\,ds}_T$ - $\mathrm{CE.}\,ds_T \geq$ ET $\qquad\qquad \forall$ T, T ∈ {sewer outlet manholes} (8-16)

$\mathrm{SE.\,ds}_T$: Surface elevation downstream for manhole at outlet manhole T.

$\mathrm{CE.}\,ds_T$: Cutting depth to downstream crown elevation manhole at outlet manhole T.

ET: The crown elevation of the existing pipe at outlet manhole T.

Equations (8-13)-(8-16) are used in both approaches. However, the second approach considers the crown elevation at the junction node to be fixed, so the values of downstream and upstream elevation at junction nodes are the same.

8.4 Application Scenarios

An example storm water network system with a simple layout is shown in Figure 8.1. The models are applied to four different scenarios of the network, taking into account changes in lengths, ground elevations, and design discharge. The reason for using different scenarios is to compare the results of commercial diameters, crown elevations, and slopes with total costs for both approaches. The example systems consider two manhole nodes m_n on INL n, two manhole nodes m_{n+1} on INL n+1, and two outlets nodes m_{n+2} on iso-nodal line n+2=1,2,3, …T, which can be expressed as an existing network or drainage area. These simple examples can be expanded to a more applicable large system. The minimum cover depth is considered to be 3 ft. Maximum and minimum velocities are 12 and 2.5 cfs, respectively, for all scenarios. Table 8.3 provides manhole information for each scenario, such as ground elevation and design flow.

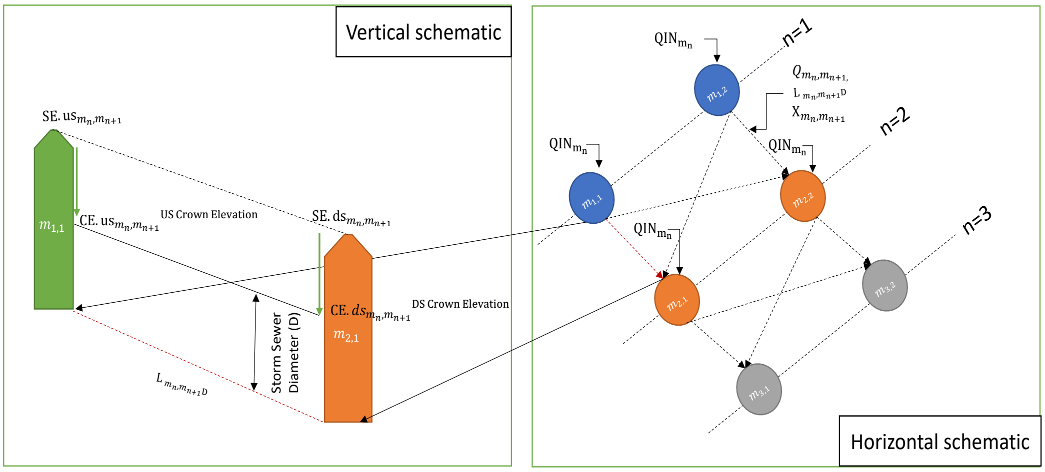


Figure 8.1. Horizontal and Vertical Schematic Setup of Four Scenario Cases of Storm Water Systems.

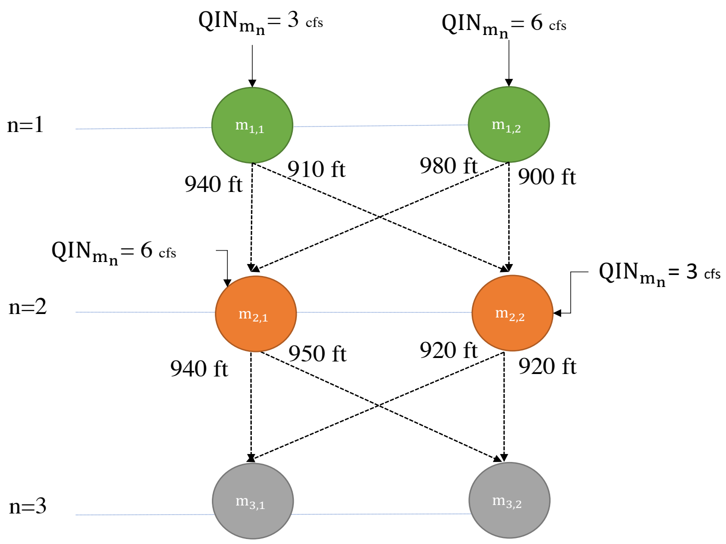
Table 8.3. Manhole Information for All Four Scenarios Systems.

Scenarios	Manhole	Ground elevation (ft)	Maximum crown elevation (ft)	Design Discharge (cfs)
S1	(3,2)	85.5	82.5	0
	(3,1)	88.5	85.5	0
	(2,2)	96.75	93.75	3
	(2,1)	92.25	89.25	6
	(1,2)	98	95	6
	(1,1)	100	97	3
S2	(3,2)	84.75	81.75	0
	(3,1)	85.5	82.5	0
	(2,2)	91.75	88.75	5
	(2,1)	90.25	87.25	5
	(1,2)	98	95	5
	(1,1)	100	97	5
S3	(3,2)	91.75	88.75	0
	(3,1)	90.5	87.5	0
	(2,2)	95.75	92.75	3
	(2,1)	94.25	91.25	0
	(1,2)	98	95	3
	(1,1)	100	97	2
S4	(3,2)	86.5	83.5	0
	(3,1)	87.25	84.25	0
	(2,2)	92.25	89.25	2
	(2,1)	90.5	87.5	5
	(1,2)	98	95	3
	(1,1)	100	97	5

8.4.1 Scenario 1

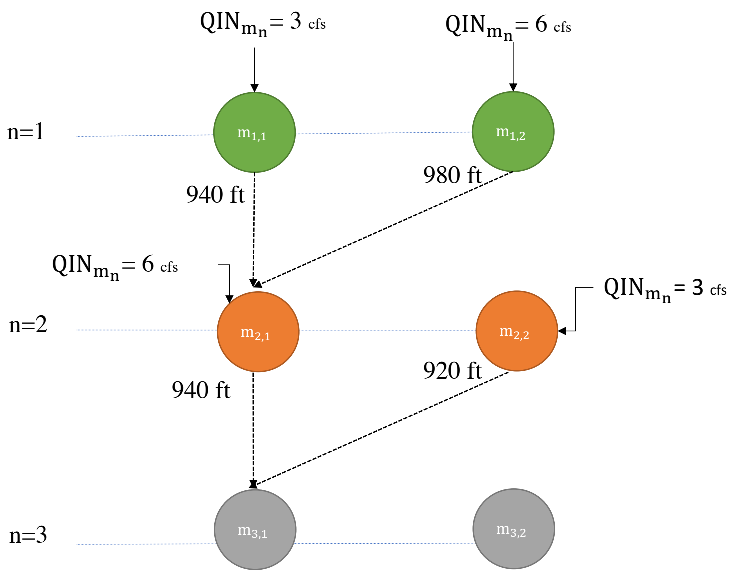
Figure 8.2.a shows the possible connections for pipes. The total distances to outlet nodes range between 2730 ft and 3810 ft. The inflow at manholes $m_{1,2}$ and $m_{2,2}$ is 6 cfs and at manholes $m_{1,1}$ and $m_{2,1}$ is 3 cfs. The downstream tie-in elevation is 80 ft. The ground elevations for each manhole are provided in Table 8.3. The minimum total costs for layout and hydraulic pipe design are $238,621 for the first approach and $249,978 for the second approach. The layout of the system is same in both approaches, but construction costs are 5% higher in the second approach since only one commercial pipe is considered per sewer pipe, as shown in Figure 8.2.b. The total length of layout is 3780 ft. The optimization models do not take the shortest length (3680 ft) due to hydraulic design constraints. For example, the ground elevation at manhole $m_{2,1}$ is 92.25 ft, while the ground elevation at manhole $m_{2,2}$ is 96.75 ft. The excavation cost at $m_{2,2}$ is higher than a longer pipeline with lower excavation costs. As shown in Table 8.4, the cutting depth to upstream manholes' crown elevation are 3 ft and 3.82 ft, respectively, for the first approach, and 3 ft and 3 ft, respectively, for the second approach. The commercial diameters of some pipes are divided into two length sections for cost minimization in the first approach, which seems to be more efficient than having one commercial diameter of that pipe. For the second approach, the commercial diameter cost is higher than the first approach, due to the consideration of a single pipe diameter. Different commercial pipe diameters were chosen for each pipe in scenario 1 as there is a different associated discharge.

a)



QIN$_{m_n}$= 3 cfs
QIN$_{m_n}$= 6 cfs
n=1
m$_{1,1}$
m$_{1,2}$
940 ft
910 ft
980 ft
900 ft
QIN$_{m_n}$= 6 cfs
QIN$_{m_n}$= 3 cfs
n=2
m$_{2,1}$
m$_{2,2}$
940 ft
950 ft
920 ft
920 ft
n=3
m$_{3,1}$
m$_{3,2}$

b)



QIN$_{m_n}$= 3 cfs
QIN$_{m_n}$= 6 cfs
n=1
m$_{1,1}$
m$_{1,2}$
940 ft
980 ft
QIN$_{m_n}$= 6 cfs
QIN$_{m_n}$= 3 cfs
n=2
m$_{2,1}$
m$_{2,2}$
940 ft
920 ft
n=3
m$_{3,1}$
m$_{3,2}$

Figure 8.2. a) Possible Connections for Pipe and b) Optimum Layout and Pipe Design for Scenario 1.

8.4.2 Scenario 2

The possible connections for pipes are shown in Figure 8.3.a. The inflow for each manhole is considered to be 3 cfs. The downstream tie-in elevation is assumed to be 80 ft. Scenario 2 is applied to identical possible connection pipes, which were considered to be 900 ft long. As a result, the minimum cost for layout and pipe design was \$240,841 for the first approach and \$244,618 for the second approach, as shown in Figures 8.3.b and 8.3.c. The commercial diameters under both approaches are same for most pipes, since the design discharges and lengths are same until they are mixed with other inflow at junction's nodes. The unique element found in scenario 2 is that, for the large discharge at manholes, it is tracks a lower ground surface to maintain smaller diameters at minimum costs or at least fewer units for larger commercial diameters for the first approach. For example, the flow between manholes $m_{2,1}$and $m_{3,1}$ is 15 cfs, so the optimization code would flow to lower ground elevation at outlet nodes to minimize the length of larger commercial diameters, as shown in Table 8.4, which could be 21" at 792 ft and 30" at 108 ft. For the second approach, the model would have a different layout and pipe design values. The model with lower values for cutting depth and where all flows would be on hydraulic performance is chosen. However, the models would reduce the total costs as much as possible.

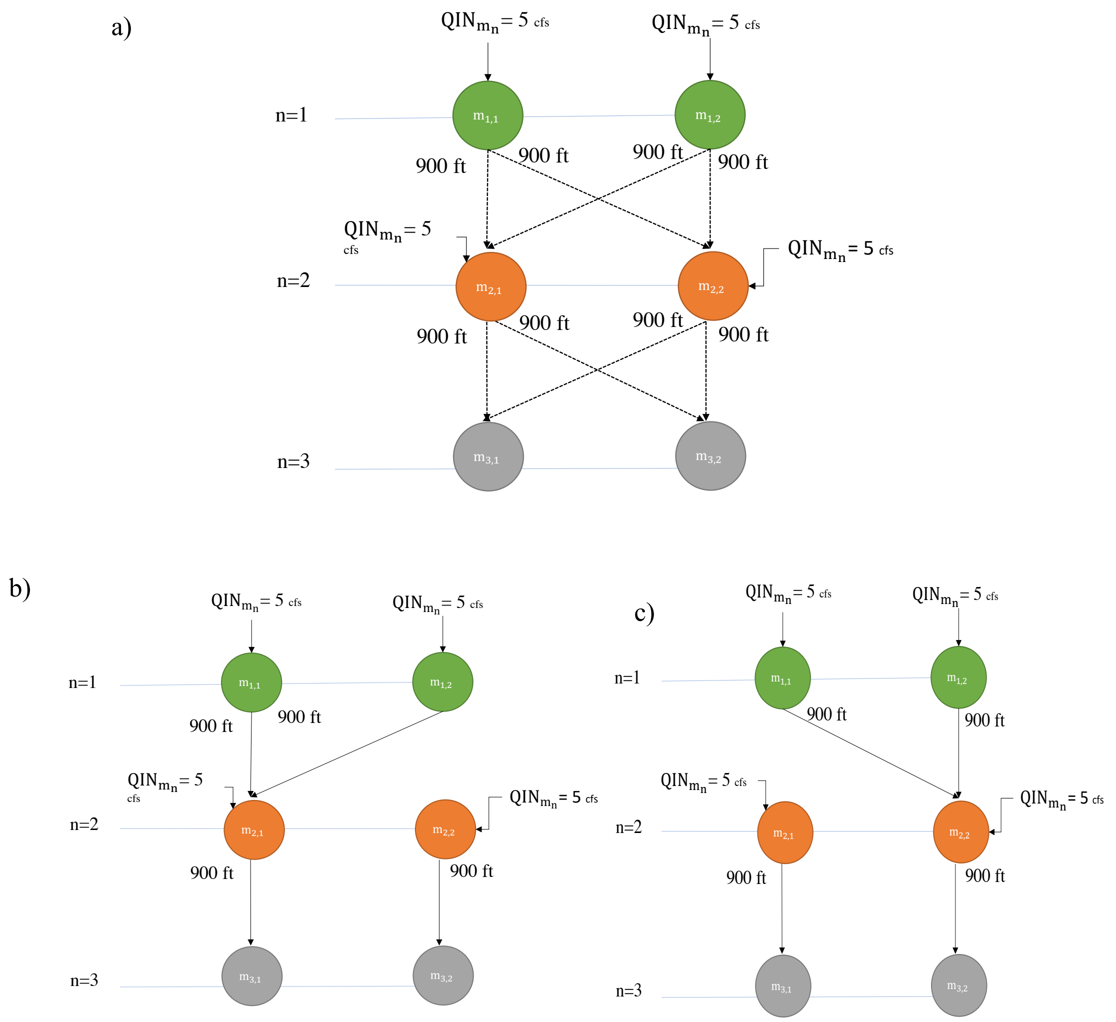


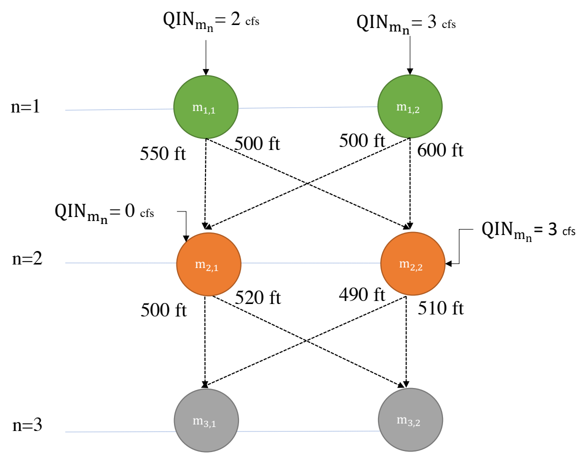
Figure 8.3. a) Possible Connections for Pipe Scenario 2, b) Layout and Pipe Design for Scenario 2-first Approach, and c) Layout and Pipe Design for Scenario 2-second Approach.

8.4.3 Scenario 3

Figure 8.4.a shows the possible connections for pipes. The total distance to outlet nodes ranges between 1560 ft and 2180 ft. The inflow at manholes $m_{1,2}$ and $m_{2,2}$ is 3 cfs and at manholes $m_{1,1}$ is 2 cfs; $m_{2,1}$ is the collection node. The layout of the system is same for both approaches, while the minimum costs for layout and pipe design are \$94,310 for the first approach and \$100,014.00 for the second approach, as shown in Figure 8.4.b. The different total costs are associated with allowing

commercial diameters and cutting depth values. The downstream tie-in elevation is considered to be 85 ft. At scenario 3, the layout selected has a minimum length of around 1590 ft, while the shortest length is 1550 ft, due to design discharges associated at manhole $m_{(2,2)}$ and no entering flows at manhole $m_{(1,2)}$. This shows the ability of the model to solve completely different design discharge systems.

a)



b)

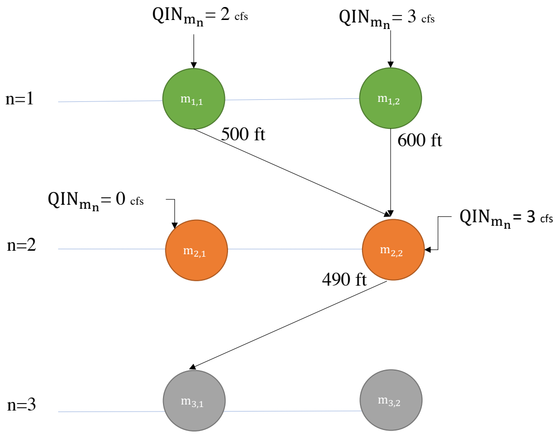
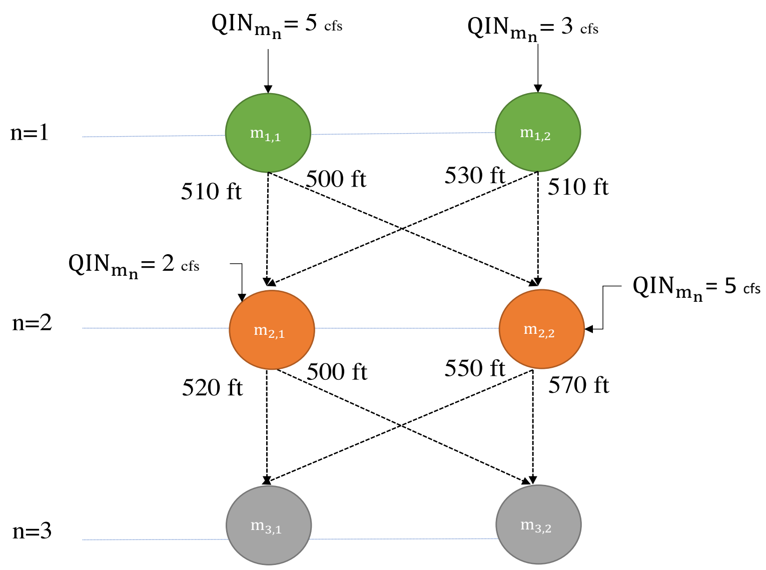


Figure 8.4. a) Possible Connections for Pipe and b) Optimum Layout and Pipe Design for Scenario 3.

8.4.4 Scenario 4

Figure 8.5.a shows the possible connections for pipes. The inflow at manholes $m_{1,1}$ and $m_{2,2}$ is 5 cfs, at manhole $m_{2,1}$ is 2 cfs, and at manhole $m_{1,2}$ is 3 cfs. The downstream tie-in elevation is 82 ft. The minimum cost for layout and pipe design is \$127,209 for the first approach and \$137,057 for the second approach, as shown in Figure 8.5.b. The cutting depth upstream is a minimum cover depth of 3 ft for each upstream manhole in the first approach and 5.10 ft at manhole $m_{2,2}$ in the second approach in order to satisfy the hydraulic design of the system. The maximum diameter is 18” for the system because of lower discharges. Interestingly, the flow from manhole $m_{1,1}$ does not go to manhole $m_{2,2}$, which has the minimum length, while the path to $m_{2,1}$ is chosen even with the pipe from manhole $m_{2,2}$ to manhole $m_{3,1}$. This is because there is a design flow at manhole $m_{2,2}$, so if it goes to $m_{2,2}$ the total flow would be 13 cfs, which requires a larger commercial diameter.

a)



b)

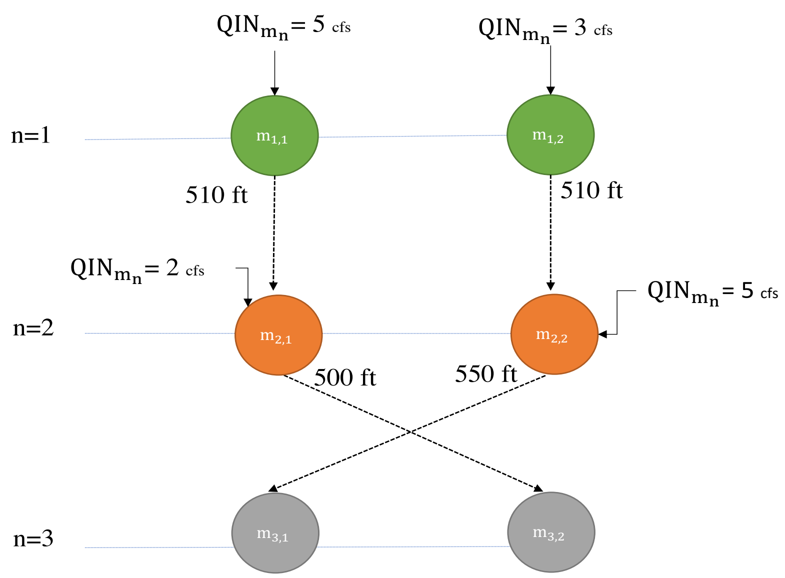


Figure 8.5. a) Possible Connections for Pipe and b) Optimum Layout and Pipe Design for Scenario 4.

8.5 Discussion of the Results

Table 8.4 provides a summary of the results of the hydraulic pipe design for all scenarios using both approaches. Crown elevations, slopes, and commercial diameters per unit length were estimated for each scenario. The results of these scenarios show that if design discharges are increased, the diameters and cost would increase, and vice versa. If the crown elevation between two INLs is increased for same design flow, the diameter and cost will also decrease, and vice versa. The length between each manhole also has a significant impact on the total cost. As shown in the results, the minimum cover depth is chosen in most cases to avoid manhole construction costs. Scenarios 3 and 4 were applied with less total length, which had a significant impact on the commercial diameter value. The commercial diameter value seems to be less compared to Scenarios 1 and 2. This type of modeling is extremely sensitive and might affect both layout and pipe design by changing one input parameter. Both approaches to commercial diameters are applied and have significantly different results in total construction costs for the system. Those savings reached up to 5% of presented small systems. These scenarios show that the ability to develop a combined model (layout and pipe design) would reduce total costs and might reach a global solution.

Table 8.4. Summary of Hydraulic Pipe Design for All Scenarios.

Scenarios	Solution approach	Segment	Upstream Node	Upstream Node	Flow	Upstream Elevation	Downstream Elevation	Cutting Depth Upstream	Cutting Depth Downstream	Length	Upstream Crown elevation	Downstream Elevations	Slope	D1		D2		Velocity		Costs
					cfs	ft	ft	ft	ft	ft	ft	ft	%	in	ft	in	ft	V (cfs)	V (cfs)	$
S1	First approach	SE1	m (1,1)	m (2,1)	3	100	92.25	3	3.819	940	97	88.431	0.009	10	163	12	777	5.50	3.82	$238,621.64
		SE2	m (1,2)	m (2,1)	6	98	92.25	3	3.819	980	95	88.431	0.007	15	627	18	353	4.89	3.40	
		SE3	m(2,1)	m (3,1)	15	92.25	88.5	3.819	8.5	940	88.431	80	0.009	21	940	---	----	6.24	---	
		SE4	m (2,2)	m (3,1)	3	96.75	88.5	3	8.5	920	93.75	80	0.015	10	619	12	301	5.50	3.82	
	Second approach	SE1	m (1,1)	m (2,1)	3	100	92.25	3	3	940	97	89.25	0.008	12	940	---	---	3.82	---	$249,978.00
		SE2	m (1,2)	m (2,1)	6	98	92.25	3	3	980	95	89.25	0.006	18	980	---	---	3.40	---	
		SE3	m (2,1)	m (3,1)	15	92.25	88.5	3	7.681	940	89.25	80.819	0.009	21	940	---	---	6.24	---	
		SE4	m (2,2)	m (3,1)	3	96.75	88.5	3	7.681	920	93.75	80.819	0.014	12	920	---	---	3.82	---	
S2	First approach	SE1	m (1,1)	m (2,1)	5	100	90.25	3	3	900	97	87.25	0.011	12	317	15	583	6.37	4.08	$240,841.28
		SE2	m (1,2)	m (2,1)	5	98	90.25	3	3	900	95	87.25	0.009	12	171	15	729	6.37	4.08	
		SE3	m (2,1)	m (3,1)	15	90.25	85.5	3	5.5	900	87.25	80	0.008	21	792	30	108	6.24	3.06	
		SE4	m (2,2)	m (3,2)	5	91.75	84.75	3	4.75	900	88.75	80	0.01	12	245	15	655	6.37	4.08	
	Second approach	SE1	m (1,1)	m (2,2)	5	100	91.75	3	3	900	97	88.75	0.009	15	900	---	---	4.08	---	$244,618.00
		SE2	m (1,2)	m (2,2)	5	98	91.75	3	3	900	95	88.75	0.007	15	900	---	---	4.08	---	
		SE3	m (2,1)	m (3,1)	5	90.25	85.5	3	3.647	900	87.25	81.853	0.006	15	900	---	---	4.08	---	
		SE4	m (2,2)	m (3,2)	15	91.75	84.75	3	4.073	900	88.75	80.677	0.009	21	900	---	---	6.24	---	
S3	First approach	SE1	m (1,1)	m (2,2)	2	100	95.75	3	3.75	500	97	92	0.010	8	44	10	456	5.732	3.669	$94,310.77
		SE2	m (1,2)	m (2,2)	3	98	95.75	3	3.75	600	95	92	0.005	12	345	15	255	3.822	2.446	
		SE3	m (2,2)	m (3,1)	8	95.75	90.5	3.75	5.5	490	92	85	0.014	15	435	18	55	6.522	4.529	
	Second approach	SE1	m (1,1)	m (2,2)	2	100	95.75	3	5.008	500	97	90.742	0.013	10	500	---	---	3.669	---	$100,014.00
		SE2	m (1,2)	m (2,2)	3	98	95.75	3	5.008	600	95	90.742	0.007	12	600	---	---	3.822	---	
		SE3	m (2,2)	m (3,1)	8	95.75	90.5	5.008	5.5	490	90.742	85	0.012	18	490	---	---	4.529	---	
S4	First approach	SE1	m (1,1)	m (2,1)	5	100	90.5	3	3	510	97	87.5	0.019	12	470	15	40	6.369	4.076	$127,209.90
		SE2	m (1,2)	m (2,2)	3	98	92.25	3	3	530	95	89.25	0.011	10	182	12	328	5.503	3.822	
		SE3	m (2,2)	m (3,1)	8	92.25	87.25	3	5.25	550	89.25	82	0.013	15	425	18	125	6.522	4.529	
		SE4	m (2,1)	m (3,2)	7	90.5	86.5	3	4.5	500	87.5	82	0.011	15	448	18	52	5.707	3.963	
	Second approach	SE1	m (1,1)	m (2,2)	5	100	92.25	3	5.106	500	97	87.144	0.02	12	500	---	---	6.369	---	$137,057.00
		SE2	m (1,2)	m (2,1)	3	98	90.5	3	3	530	95	87.5	0.014	12	530	---	---	3.822	---	
		SE3	m (2,2)	m (3,1)	10	92.25	87.25	5.106	5.106	550	87.144	82.144	0.009	18	550	---	---	5.662	---	
		SE4	m (2,1)	m (3,2)	5	90.5	86.5	3	3	500	87.5	83.5	0.008	15	500	---	---	4.076	---	

8.6 Summary and Conclusion

Optimization models are developed to simultaneously determine the layout and pipe design for storm water systems. The pipe design process includes the computation of commercial diameters, pipe slopes, and crown elevations for storm water pipes. Optimization models aim to minimize the total costs of the layout and pipe design for most system elements. The models are based on two different approaches: 1) one that considers one or more commercial diameters and 2) one that only considers one commercial diameter for a pipe connecting two manholes. The commercial diameters, pipe slopes, crown elevations, and total costs of the storm sewer system were obtained via the first approach and compared to commercial diameters, pipe slopes, crown elevations, and total costs of the storm sewer system obtained from the second approach. Optimization models are able to design the hydraulics and layout of storm sewer systems simultaneously. Previous work by Karovic and Mays (2014) and Steele et al. (2016) showed that the problem of layout and pipe design of storm sewers can reduce total costs by 9%. The model for optimal layout and pipe design storm sewer systems presented here using MINLP was developed in GAMS. However, the total cost of having only one commercial diameter is slightly more than having two commercial diameters per unit length. The model goes a step further than previous models, where the authors included hydraulic pipe design with layout design in one optimization model. Steele et al. (2016) described the advantage of using optimization models to design sewer and sanitary systems. The design flows do not necessarily select the shortest length, but they also consider other factors, such as hydraulic and topography constraints of the case study. However, combining layout and pipe design in one optimization model should be cheaper than using two optimization models separately. Developing these types of models would help construct a storm water system that ensures all design parameters at minimum cost. This chapter presented a new approach for solving commercial diameter and sewer layout problems. Based on these scenario applications, the model can be extended to larger systems by introducing more INLs.

8.7 Notation

Sets

n: Set number of INLs

m_n: Set of manhole nodes on INL n.

m_{n+1}: Set of manhole nodes on INL n+1.

T: Set of sewer outlet manholes.

D: Set of candidate pipe diameter (8”, 10”, 12”, 15”, ….30”) in inches.

Parameters

C_D: Cost of commercial diameter per unit length.

C_{n,m_n}: Costs of manhole m_n associated with cutting depth to upstream crown elevation.

$SE.us_{n,m_n,m_{n+1}}$: Surface elevation upstream for manhole m_n on INL n to manhole m_{n+1} and INL n+1.

$SE.ds_{n,m_n,m_{n+1}}$: Surface elevation downstream for manhole m_n on INL n to manhole m_{n+1} on INL n+1.

$SE.ds_T$: Surface elevation downstream for manhole at outlet manhole T.

ET: Crown elevation of existing pipe at outlet manhole T.

QIN_{n,m_n}: Surface inflow into manhole m_n on INL n.

SloMax: Maximum slope of 2% for candidate commercial diameters.

SloMin: Minimum slope of 0.5% for candidate commercial diameters.

$\mathrm{m_D}$: Constant value 2.16 for U.S. units (3.21 for SI units).

BigM: Plenty value = 10^{10}.

Variables

State variables:

$\mathrm{Q}_{n,\mathrm{m_n},\mathrm{m_{n+1}},}$: Flow through pipes between manhole $\mathrm{m_n}$ on INL n to manhole $\mathrm{m_{n+1}}$ and INL line n+1.

$\mathrm{L_{n,m_n,\,m_{n+1},D}}$: Length of pipe of diameter D connecting manhole $\mathrm{m_n}$ on INL n to manhole $\mathrm{m_{n+1}}$ on INL n+1.

$\mathrm{C\,E.\,us_{n,m_n,m_{n+1}}}$: Cutting depth to upstream crown elevation for manhole $\mathrm{m_n}$ on INL n to manhole $\mathrm{m_{n+1}}$ on INL n+1.

$\mathrm{C\,E.\,ds_{n,m_n,m_{n+1}}}$: Cutting depth to downstream crown elevation for manhole $\mathrm{m_n}$ on INL n to manhole $\mathrm{m_{n+1}}$ on INL n+1.

$\mathrm{CE.}\,ds_T$: Cutting depth to downstream crown elevation manhole at outlet manhole T.

$\mathrm{Slope_{n,m_n,m_{n+1}}}$: Slope of pipes between $\mathrm{m_{n-1}}$ on INL n-1 to manhole $\mathrm{m_n}$ and INL n.

Decision variables:

$\mathrm{X_{n,m_n,m_{n+1}}}$: 0-1 binary variable, where 1 indicates a pipe connection between manholes $\mathrm{m_n}$ and $\mathrm{m_{n+1}}$, otherwise 0.

$\mathrm{CanD_{n,m_n,m_{n+1},D}}$: 0-1 binary variable, where 1 indicates only one commercial diameter between manholes $\mathrm{m_n}$ and $\mathrm{m_{n+1}}$, otherwise 0.

9 STORM SEWER DESIGN USING NONLINEAR PROGRAMMING

9.1 Introduction

The objectives of this chapter are threefold. (1) This chapter presents a new technique developed in the GAMS program to find the optimum solution for pipe design (crown elevations, pipe slopes, and commercial diameters) for storm sewer systems using an NLP procedure. (2) It explains the model development of the NLP optimization approach. (3) It also applies the model to the storm sewer system published by Karovic and Mays (2014) using SA and comparing the results.

9.2 Previous Models

The first optimal sewer design was proposed the in early 1960s (Deininger, 1966; Holland, 1966). From 1970 to 1980, simulation and optimization models were developed for cost-effective design. Many different optimization techniques were developed, such as LP, NLP, DDDP, DP, GAs, and SA. These included the approaches developed by Akfirat and Deininger, 1966, Argaman, Shamir, and Spivak, 1973, Afshar, 2010, Afshar et al., 2006, Barlow, 1972, Brown and Koussis, 1987, Dajani and Hasit, 1974, Elimam, Charalambous, and Ghobrial, 1989, Farmani, Savic, and Walters, 2006, Gidley, 1986, Haghighi, 2013, 2017, Haghighi and Bakhshipour, 2016, Izquierdo et al., 2008, Karovic and Mays, 2014, Lowsley, 1973, Mays, 1976, Mays, Liebman, and Wenzel, 1976, Mays and Yen, 1975, Meredith, 1971, Miles and Heaney, 1988, Moeini and Afshar, 2012, Navin and Mathur, 2016, Nzewi, Gray, and Houck, 1985, Pan and Kao, 2009, Price, 1978, Steele et al., 2016, Walters and Lohbeck, 1993, Walters and Smith, 1995, Walters and Templeman, 1979, Yang and Su, 2007, Yen, 2001, and Yen et al., 1984. Karovic and Mays (2014) discussed in more detail the different optimization methods used for optimal layout and pipe design.

9.3 Optimization Approach

The new approach identifies the crown elevations from a different perspective, considering the depth to the upstream crown elevation from the ground surface. In the optimization model, two decision variables are considered: 1) commercial pipe diameters and 2) upstream crown elevations. However, two main constraints used in the model for pipe design purposes are length constraints and hydraulic constraints. The length for a specific pipe diameter is a decision variable and the size of the diameter of the pipe is commercial. The advantages of using length constraints include avoiding continuous values for diameters and introducing cost values for each commercial diameter per unit length.

To design any water network system, the hydraulic constraints are used to guarantee the flows in the network, comprised of minimum and maximum values. These values are defined through Manning's equation or Darcy-Weisbach's equation, which consider the appropriate values for diameters, slopes, and discharges. However, Manning's equation was modified for this mathematical formulation, which solved for hydraulic constraints (Mays, 2011).

$$E2 - E1 = \left(\frac{{m_D}^2 Q^2 n^2}{D^{\frac{16}{3}}}\right) L \qquad (9\text{-}1)$$

Where $m_D$is 2.16 for U.S. units (3.21 in SI units), Q is flow rate (cfs in U.S. units or m3/s in SI metric units). D is rounded up to the next commercial size pipe (pipe diameter in m or in), and n is Manning's value, which depends on the pipe material. S_0 is the slope of hydraulic grade line, dimensionless S = $E_2 - E_1/L$, where

E_1 is the crown elevation downstream, E_2 is the crown elevation upstream, and L is the length of pipe in feet or meters.

The NLP optimization method presented here was developed in GAMS. The problem is transferred from LP to NLP because of the constraint number (9). The Linear Interactive and Discrete Optimizer (LINDO) solver is supported and used to solve nonlinear problems. The first step was to summarize all input data and table information for the network into sets and parameters. The optimizer consists of an NLP approach, along with hydraulic computations that were developed using Manning's equations, as shown in (Eq. 1). Other constraints, such as continuous slope, minimum cover depth, sewer at the junction point, and downstream tie-in elevation, are defined as depth at an upstream crown elevation $\mathrm{CE.us}_{n,m_n,m_{n+1}}$. The model starts initial solutions for a new depth at the upstream crown elevation for each pipe segment in the network. Once the new crown elevations are obtained, Manning's equation is used to check whether the new set of crown elevations is feasible or not. The feasibility of new crown elevations was obtained using cost penalties that eliminate solutions that did not satisfy velocity and diameters for downstream direction constraints. For example, if the new crown elevations for the pipe segment resulted in velocity over a maximum limit for partial candidate diameters, and the penalty costs for those partial candidate diameters were applied until the commercial diameters fit the new crown elevations. The commercial diameter for downstream direction must be equal to or greater than the commercial diameter upstream. If the commercial diameter downstream size is smaller than the commercial diameter upstream, apply penalty costs for that commercial diameter. The penalty costs are applied for only two constraints included in the objective function, while the other constraints are used as is in the optimization code.

The main goal of the model is to present a new technique that can be used to find the optimum pipe design for the collection type network. Indeed, the objective function is to minimize the total costs for pipe design of the system's elements associated with commercial diameters, pipe slopes, and crown elevations for given layouts.

9.4 Model Formulation

The optimal design of a storm water system for given layouts determines the commercial pipe diameter, pipe slopes, and crown elevations of the network for minimum total costs. Figure 9.1 shows the significant detailed components of storm sewer segments in the system. The INL method is used as a principle concept of the model to solve pipe design and saves costs for the collection type system.

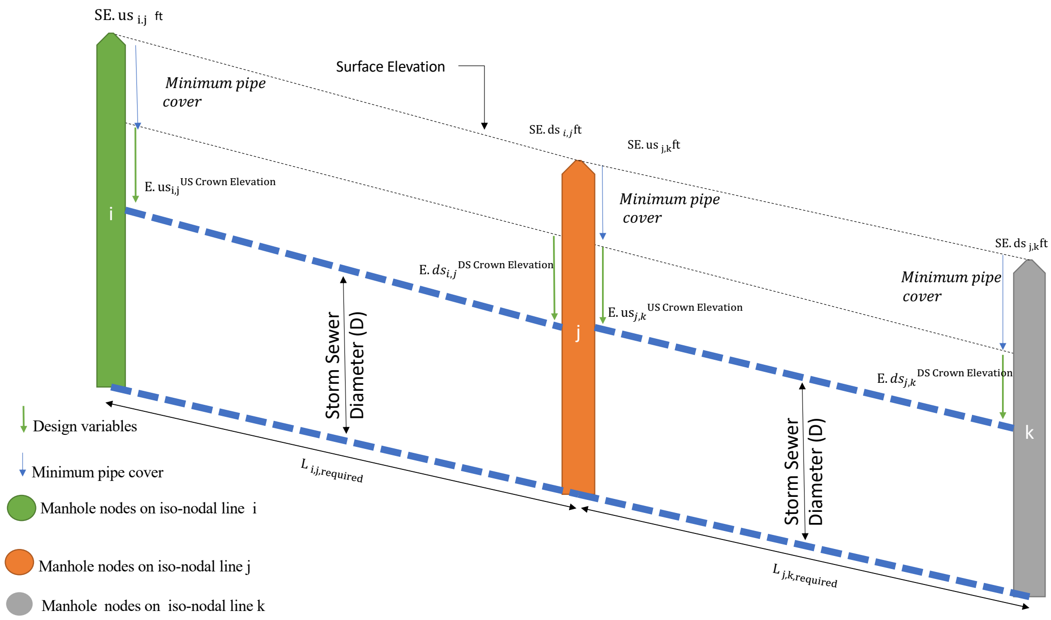


Figure 9.1 The Detailed Major Components of Storm Sewer Segments

Minimizing the total costs of commercial diameters in length $L_{n, m_n, m_{n+1}, D}$ and manhole costs associated with the depth to upstream crown elevation.

$$\text{Min } \sum_n \sum_{m_{n+1}} \sum_{m_n} \left(\left(\sum_D C_D \, L_{n,m_n, m_{n+1}, D} \right) + C_{n,m_n} \, CE.us_{n,m_n,m_{n+1}} \right) \qquad (9\text{-}2)$$

Where C_{n,m_n} represents the manhole costs per unit depth of pipe (n, n+1). $CE.us_{n, m_n, m_{n+1}}$ is the depth to upstream elevation of pipe on INLs (n, n+1). C_D is the cost of commercial diameters per unit length of pipe connecting INLs n and n+1. $L_{n, m_n, m_{n+1}, D}$ is the length of pipe for diameter D connecting manhole m_n on INL n to manhole m_{n+1} on INL n+1 to manhole m_{n+2} on INL n+2.

Subject to hydraulic constraints:

1- Storm sewer hydraulics defined by Manning's equation for each manhole m_n to manhole m_{n+1}

$$\sum_D \frac{{m_D}^2 Q^2_{n,m_n, m_{n+1}} n^2}{{D_{n,m_n,m_{n+1}}}^{\frac{16}{3}}} L_{n,m_n, m_{n+1}, D}$$
$$= ((SE.us_{n,m_n,m_{n+1}} - CE.us_{n,m_n,m_{n+1}}) - (SE.ds_{n,m_n,m_{n+1}} - CE.ds_{n,m_n,m_{n+1}}))$$
$$\forall\, n, \forall\, m_n, \forall\, m_{n+1} \qquad (9\text{-}3)$$

Where D is the pipe diameter (in), n is Manning's roughness coefficient, $CE.ds_{n,m_n,m_{n+1}}$ is the depth to downstream crown elevation for pipe on INLs (n, n+1), $CE.us_{n,m_n,m_{n+1}}$ is the depth to the upstream elevation of pipe on INLs (n, n+1), $Q_{n,m_n,m_{n+1}}$ is the design flow for pipe on INLs (n, n+1), and $L_{n,m_n,m_{n+1},D}$ is the length of pipe for diameter D for pipe on INLs (n, n+1).

2- Continues slope constraints to ensure that slope continues downstream.

$$CE.ds_{n,m_n,m_{n+1}} \geq CE.us_{n,m_n,m_{n+1}} \tag{9-4}$$

3- Sewers are joined at junctions such that upstream crown elevation of the downstream sewer $CE.us_{n,m_n,m_{n+1}}$ is greater than the downstream crown elevation of the downstream sewer $CE.ds_{n,m_{n-1},m_n}$

$$CE.us_{n,m_n,m_{n+1}} \geq CE.ds_{n,m_{n-1},m_n} \ \forall n, \forall m_{n-1}, \forall m_n, \forall m_{n+1} \tag{9-5}$$

4- The depth should take into account the minimum cover depth of the ground surface elevation to protect the pipe from the damage of heavy traffic loads. Minimum cover depth for scenarios is considered to be 3 ft.

$$CE.us_{n,m_n,m_{n+1}} \geq Min_{coverdepth} \ \forall n, \forall m_n, \forall m_{n+1} \tag{9-6}$$

5- Tie-ins to existing sewer systems must also be defined if the sewer being designed connects to outlet sewer; thus, the new crown elevation must be lower than the crown elevation of the existing pipe ET.

$$SE.ds_T - CE.ds_T \geq ET \qquad \forall T, T \in \{\text{sewer outlet manholes}\} \tag{9-7}$$

6- Length constraints for each link force the sum of lengths for certain diameters to be equal to the total reach length required. Further, it can consider more than one diameter for each pipe linking between manholes m_n, m_{n+1}. The length required is known value.

$$\sum_D L_{n,m_n,\, m_{n+1},D} \; = L_{\, n,m_n,\, m_{n+1} required} \;\; \forall\, n \,, \forall\, m_n, \forall\, m_{n+1} \qquad (9\text{-}8)$$

A signing one commercial diameter for each pipe connecting manholes between INL n and INL n+1 is expressed as the following:

$$\sum_D L_{\, n,\, m_n, m_{n+1},D} \,/\, L_{\, n,\, m_n, m_{n+1},D} \;\leq 1 \qquad \forall\, n \,, \forall\, m_n, \forall \;\; m_{n+1} \qquad (9\text{-}9)$$

However, the constraint will transfer the problem into non-linear form. Also, it has the ability to a sign only one commercial diameter.

7- The new storm sewer cannot have pipe segments that decrease in diameter in the downstream direction, i.e., the constraint does not allow water to flow from a flat pipe with a larger diameter to a steep pipe with a small diameter.

$$D_{n,m_n,m_{n+1}} \geq \; D_{n,m_{n-1},m_n} \; \forall\, n \,, \forall \;\; m_{n-1}, \forall \;\; m_n, \forall \;\; m_{n+1} \qquad (9\text{-}10)$$

Pipe velocity must be less than the maximum permissible velocity (to prevent the effects of high velocity flows) and greater than the minimum permissible velocity (to prevent deposition).

$$V_{max} \geq V_{n,\, m_n, m_{n+1}} \geq V_{min} \; \forall\, n,\; m_n, m_{n+1} \qquad (9\text{-}11)$$

Equations (10) and (11) have constraints included in the objective function using the penalty method.

$$V_{(n,m_n,m_{n+1})} = \begin{cases} V_{n,m_n,m_{n+1}} BigM & , \quad V_{n,m_n,m_{n+1}} < V_{min} \\ V_{n,m_n,m_{n+1}} & , \quad V_{n,m_n,m_{n+1}} \geq V_{min} \end{cases}$$

$$V_{(n,m_n,m_{n+1})} = \begin{cases} V_{n,m_n,m_{n+1}} & , \quad V_{n,m_n,m_{n+1}} < V_{max} \\ V_{n,m_n,m_{n+1}} BigM & , \quad V_{n,m_n,m_{n+1}} \geq V_{max} \end{cases}$$

$$D_{(n,m_n,m_{n+1})} = \begin{cases} D_{n,m_n,m_{n+1}} \text{BigM}, & D_{n,m_n,m_{n+1}} < D_{n,m_{n-1},m_n} \\ D_{n,m_n,m_{n+1}}, & D_{n,m_n,m_{n+1}} \geq D_{n,m_{n-1},m_n} \end{cases}$$

Where BigM has a penalty value $=10^{10}$. Therefore, if the pipe segment meets the min and max allowable velocities, it will not apply penalty costs. But if the pipe segment does not meet the min and max allowable velocities, it will apply a cost penalty for value BigM. If the commercial diameter stays the same or increases in size in the downstream direction, it will not apply penalty costs. If the commercial diameter downstream pipe is smaller than the commercial diameter upstream pipe, it will apply a cost penalty for value BigM.

The only design variables for the optimization procedure (GAMS) is the depth of crown elevation of the upstream pipe and length for certain diameter pipes in each of these links.

9.5 Cost Functions

The cost functions of commercial diameter, manhole, and permanent pavement replacement were developed by Karovic and Mays (2014). The costs of commercial pipe diameters vary from 24” to 84”, which is provided as a function of unit length. The cost of permanent pavement replacement is \$40 per square yard, which is calculated by taking into account a two-foot plus commercial diameter size for the width of the trench. Manhole costs are defined as a function of depth to upstream crown elevation $CE.us_{n,m_n,m_{n+1}}$. The manhole cost function was developed after using a regression analysis in Excel. Total costs of commercial diameter and manholes are as shown in Table 9.1 and Figure 9.2.

Cost(\$/depth) = 1818.2 (depth to crown Elevation @ upstream-ft) -1000

Table 9.1. Example Pipe Costs

Pipe Cost			
(in)	($/ft)	Permanent Pavement Replacement ($/ft)	Final Total Costs ($/ft)
24	80	17	97
30	90	20	110
36	115	22	137
42	140	24	164
48	155	26	181
54	205	28	233
66	250	33	283
72	295	35	330
84	355	40	395

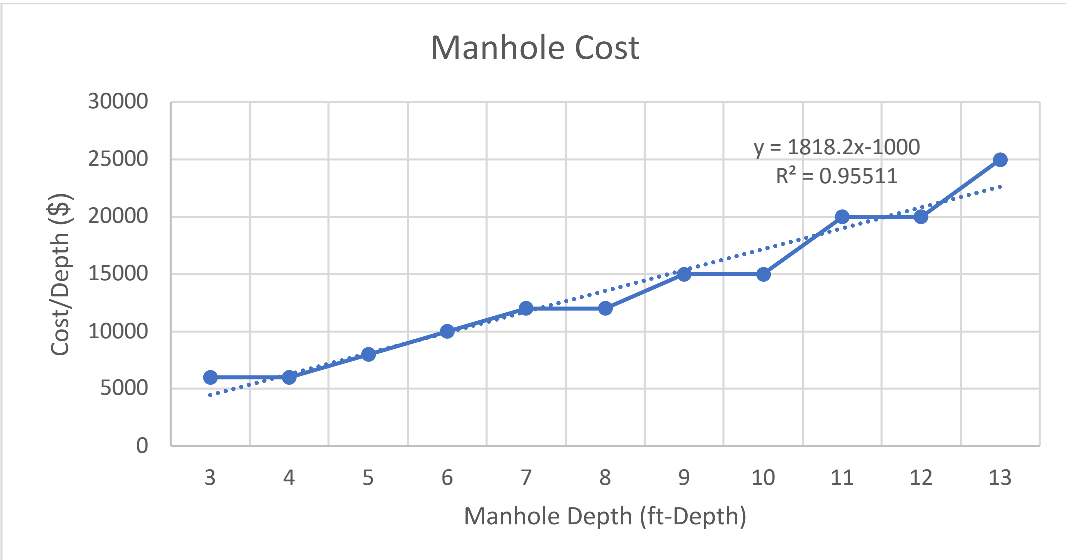


Figure 9.2. Manhole Cost per Unit Depth (adapted from Karovic & Mays,2014).

9.6 Example 1

Example 1 is partial system of a network by Karovic and Mays (2014), who applied a GAMS model as the first step to include all system elements. The system contains seven manhole nodes connected to each other (MH-L7 > MH-L6 > MH-L5 > MH-L4 > MH-L3 > MH-L2 > MH-L1). Here, MH-L1 is assumed to be the outlet manhole that is connected to an existing network, with an elevation of 1242 ft. The data for the system is provided in Table 9.2 and Figure 9.3.

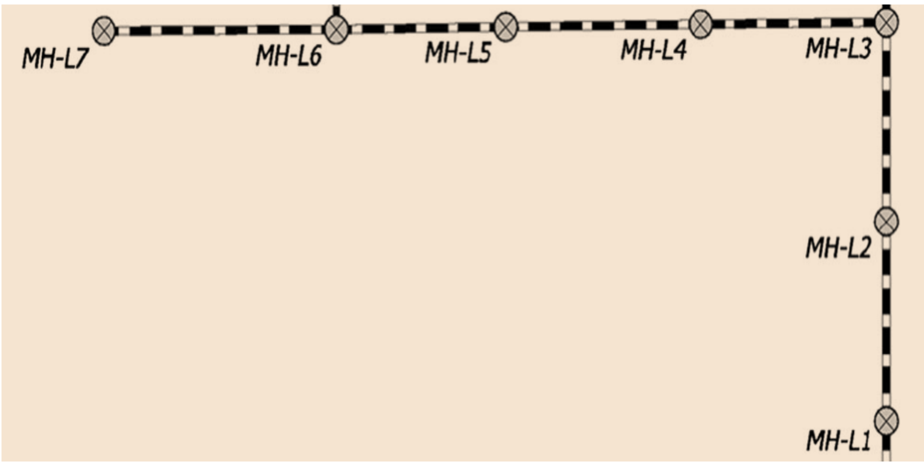


Figure 9.3. Horizontal Section for Example 1.

Table 9.2. Data Information for Example 1.

Segment	Upstream Manhole	Downstream Manhole	Pipe Flow (cfs)	Segment Length (ft)	Upstream Rim Elevation (ft)	Downstream Rim Elevation (ft)
L2	MH-L2	MH-L1	52.5	389	1248.64	1247.54
L3	MH-L3	MH-L2	52.5	392	1251.13	1248.64
L4	MH-L4	MH-L3	32.6	441	1252.59	1251.13
L5	MH-L5	MH-L4	32.6	464	1254.36	1252.59
L6	MH-L6	MH-L5	32.6	403	1256.81	1254.36
L7	MH-L7	MH-L6	15.1	553	1257.93	1256.81

9.6.1 Results of Example 1

Table 9.3 shows the total costs for Example 1 using the optimal NLP design procedure. Since the flow increases downstream, the slopes increase. The total cost for the partial section of the system is $369,499. The commercial diameters increase since the design flow increases, which sometimes needs to be adjusted by using the plenty cost method of the velocity function to avoid smaller diameters downstream.

The main goal of implementing a partial section of the network is to check validity and enhance it using the model for the whole network.

Table 9.3. Optimization Results for Example 1.

Storm Sewer Segment	Upstream Crown Elev.	Downstream Crown Elev.	Slope	Diameter	Costs ($)		Total Costs
	(ft)	(ft)	(%)	in	Manhole Cost	Diameter Costs	($)
L2	1244.47	1243.41	0.0027	42.0	6578.3	63796.0	$ 70374.3
L3	1245.54	1244.47	0.0027	42.0	9163.7	64288.0	$ 73451.7
L4	1248.33	1245.54	0.0063	30.0	6749.2	48510.0	$ 55259.2
L5	1251.26	1248.33	0.0063	30.0	4632.8	51040.0	$ 55672.8
L6	1253.81	1251.26	0.0063	30.0	4454.6	44330.0	$ 48784.6
L7	1254.56	1253.81	0.0014	30.0	5127.3	60830.0	$ 65957.3

9.7 Example 2

Example 2 considers storm sewer system network given by Karovic and Mays (2014), as shown in Figure 9.4. In the example system, MH-S0 is the outlet manhole connected to an existing sewer network at an elevation of 1239 ft. The data for the sewer system is provided in Table 9.4. More detailed description of the network can be found in Karovic and Mays (2014).

Table 9.4. Data Information for Example 2.

Segment	US Manhole	DS Manhole	Pipe Flow (cfs)	Segment Length (ft)	Upstream Elevation (ft)	Downstream Elevation (ft)
S1	MH-S1	MH-S0	108.5	70	1244.74	1243.11
S2	MH-S2	MH-S1	108.5	623	1246.43	1244.74
S3	MH-S3	MH-S2	38.1	340	1246.8	1246.43
S4	MH-S4	MH-S3	38.1	114	1247.64	1246.8
S5	MH-S5	MH-S4	38.1	77	1247.86	1247.64
S6	MH-S6	MH-S5	28.5	358	1249.2	1247.86
S7	MH-S7	MH-S6	28.5	434	1250.93	1249.2
S8	MH-S8	MH-S7	13.4	72	1251.12	1250.93
S9	MH-S9	MH-S8	13.4	257	1251.59	1251.12
L1	MH-L1	MH-S2	52.5	367	1247.54	1246.43
L2	MH-L2	MH-L1	52.5	389	1248.64	1247.54
L3	MH-L3	MH-L2	52.5	392	1251.13	1248.64
L4	MH-L4	MH-L3	32.6	441	1252.59	1251.13
L5	MH-L5	MH-L4	32.6	464	1254.36	1252.59
L6	MH-L6	MH-L5	32.6	403	1256.81	1254.36
L7	MH-L7	MH-L6	15.1	553	1257.93	1256.81
A1	MH-A1	MH-L3	12.4	332	1252.79	1251.13
A2	MH-A2	MH-A1	12.4	220	1254.01	1252.79
B1	MH-B1	MH-L6	11.2	267	1257.34	1256.81
B2	MH-B2	MH-B1	11.2	404	1257.76	1257.34

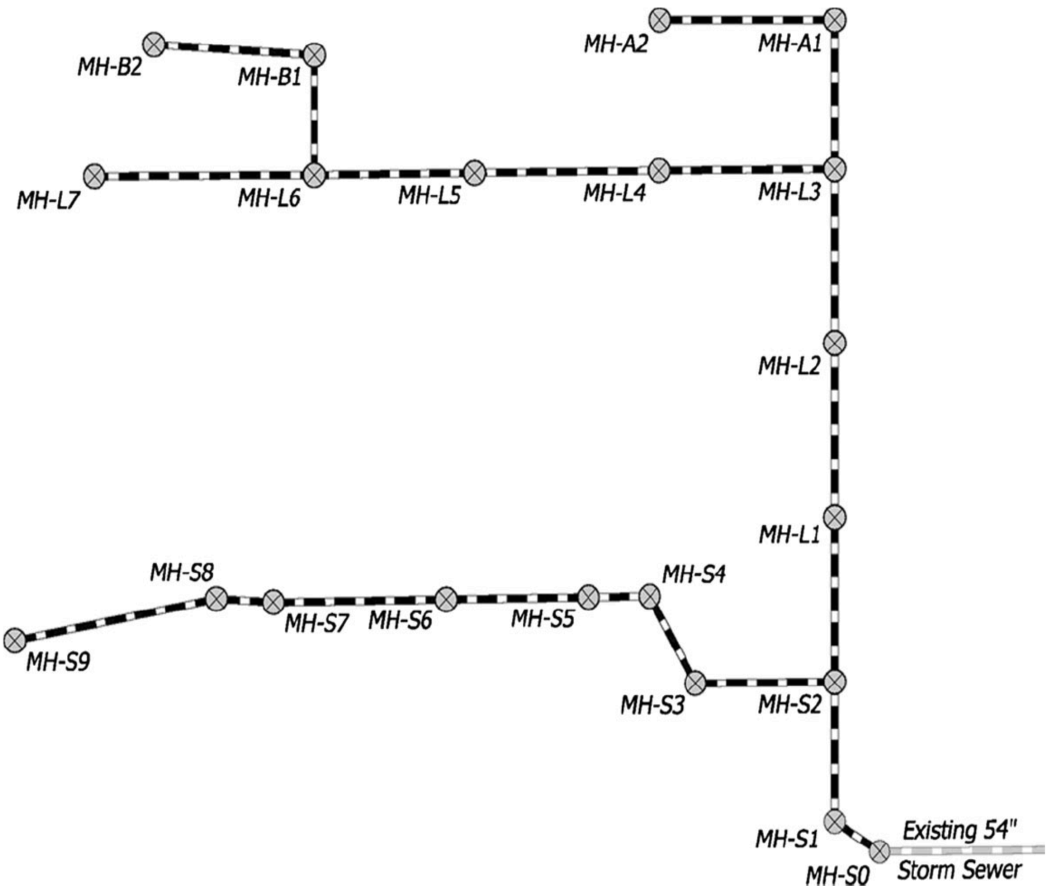


Figure 9.4. Example 2 from Karovic and Mays' (2014) Network.

9.7.1 Results of Example 2

The comparison of total costs of the storm water system obtained by the SA design method and the NLP method design is presented in Table 9.5 and Figure 9.4. The optimization model was formulated using NLP in GAMS. The construction costs for the system using the optimization model was $963,388, which is 7% lower than the optimal SA design and 16% lower than using conventional design procedures. The total saving costs are visible for most segments, except segment S6, since the commercial diameters increased more than in the optimal SA design. Significant cost savings are obtained based on manhole costs. This is because the objective functions have the manhole costs as a function of the depth upstream of each segment.

Furthermore, the height cost savings are shown in segment S2, which is considered to be the most significant segment length. However, the depth for all crown elevations upstream is between 3ft to 5 ft, which reduces the total costs by using this approach. Manhole costs were also reduced by using a function of depth for crown elevations upstream only. The commercial diameters are similar in most segments, except for segment S6. However, the model presents a new technique for solving optimal pipe design of storm water system design. One commercial diameter constraint for each pipe was applied in order to use a similar approach to the SA model for determining commercial diameters. Moreover, the model was able to consider more than one commercial diameter in one pipe, which can reduce the total costs of the network.

Table 9.5. Optimization Design Procedure Cost Comparison Using SA vs. NLP.

Segment	Optimal SA Design					Optimal NLP Design					
	US Crown Elev.	DS Crown Elev.	Pipe Slope	Pipe Diameter	Segment Cost	US Crown Elev.	DS Crown Elev.	Pipe Slope	Pipe Diameter	Segment Cost	Cost Saving
S1	1239.4	1239.05	0.00495	48	$27,717	1239.70	1239.30	0.0057	48	$20,827	$6,890
S2	1242.61	1239.4	0.00515	48	$125,178	1243.26	1239.70	0.0057	48	$117,525	$7,653
S3	1243.33	1242.61	0.00211	42	$65,911	1243.80	1243.31	0.0014	42	$60,215	$5,696
S4	1243.71	1243.33	0.00336	36	$25,643	1244.17	1243.80	0.0033	36	$20,923	$4,720
S5	1244.02	1243.71	0.00405	36	$20,566	1244.68	1244.43	0.0033	36	$15,325	$5,241
S6	1245.5	1244.02	0.00413	30	$49,380	1245.34	1244.68	0.0018	36	$55,069	$0
S7	1247.38	1245.5	0.00434	30	$57,740	1247.43	1245.34	0.0048	30	$53,095	$4,645
S8	1247.65	1247.38	0.00375	24	$15,040	1247.69	1247.43	0.0035	24	$12,225	$2,815
S9	1248.6	1247.65	0.00368	24	$31,129	1248.59	1247.69	0.0035	24	$29,384	$1,745
L1	1243.94	1242.61	0.00363	42	$72,351	1244.26	1243.26	0.0027	42	$65,149	$7,202
L2	1245.24	1243.94	0.00333	42	$73,969	1245.32	1244.26	0.0027	42	$68,830	$5,139
L3	1247.71	1245.24	0.00632	36	$63,791	1247.75	1245.32	0.0062	36	$58,845	$4,946
L4	1248.99	1247.71	0.00291	36	$70,515	1248.81	1247.75	0.0024	36	$66,296	$4,219
L5	1250.34	1248.99	0.00289	36	$75,671	1249.92	1248.81	0.0024	36	$70,648	$5,023
L6	1252.67	1250.34	0.00579	30	$54,330	1252.46	1249.92	0.0063	30	$51,232	$3,098
L7	1254.93	1252.67	0.00408	24	$62,071	1254.93	1252.46	0.0045	24	$58,096	$3,975
A1	1248.9	1247.71	0.00357	24	$40,462	1249.13	1248.13	0.0030	24	$37,862	$2,600
A2	1251.01	1248.9	0.00961	24	$27,511	1250.45	1249.79	0.0030	24	$26,810	$701
B1	1253.56	1252.67	0.00331	24	$34,107	1253.77	1253.11	0.0025	24	$31,392	$2,715
B2	1254.73	1253.56	0.00291	24	$47,502	1254.76	1253.77	0.0025	24.00	$43,643	$3,859
					$1,040,584					$963,388	$77,195.54

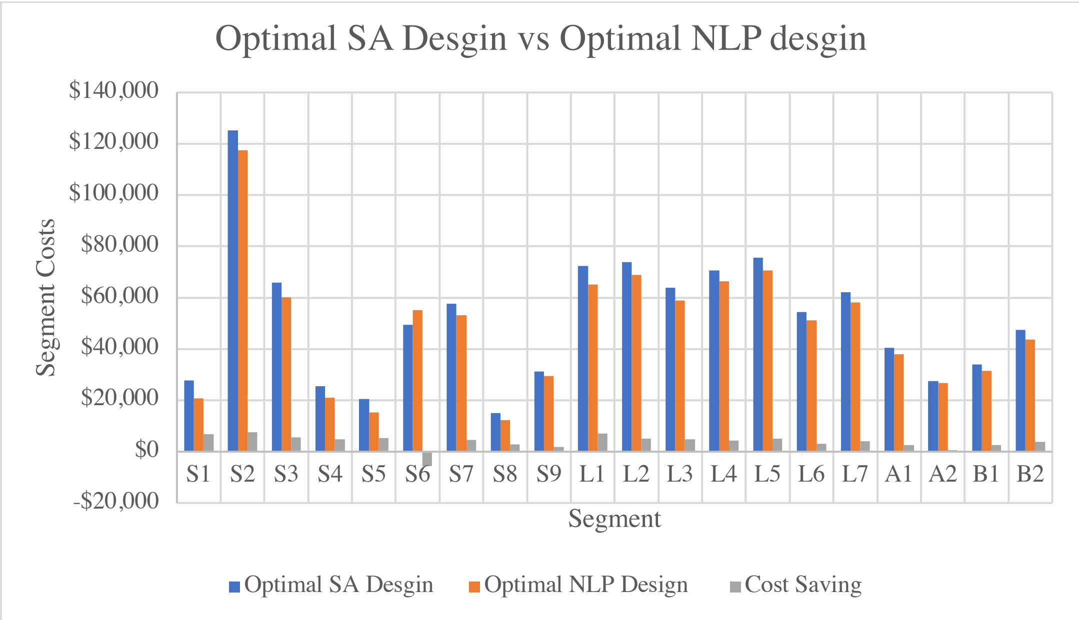


Figure 9.5. Optimal SA Design vs. Optimal NLP Design.

9.8 Conclusion

The optimization model was developed to solve the hydraulic design of the storm water system for given layouts. The hydraulic design process included computation of commercial diameters, pipe slopes, and crown elevations for storm water pipes. The optimization model was formulated as an NLP problem and was solved using GAMS. The applied model was derived from the storm sewer system published by Karovic and Mays (2014). The total costs for commercial diameters, crown elevations, and pipe slopes of the storm sewer system were obtained using the optimal design NLP procedure. These were compared to the total costs of commercial diameters, crown elevations, and pipe slopes of the storm sewer system obtained from SA. The results show that the model reduced more than 7% of the total costs of the SA approach. The significant savings were obtained from lower manhole costs for most segments in the network. However, the optimization technique tool was used to solve many storm water system designs. Unfortunately, some design practices do not use this tool for solving problems.

Moreover, many scientific papers have proven the power of optimization models, which can save up to 30% of total costs. This chapter showed that different solution methods can lead to savings of up to 16% compared to using conventional design procedures. By using an optimization approach, the total cost of the storm sewer system was reduced from $1,117,700 to $963,388, for a total savings of over $154,321, or about 14%. The costs would have been reduced even further if the model had taken into account more than one commercial diameter for each connected pipe.

9.9 Notation

Sets

n: Set number of INLS.

m_n: Set of manhole nodes on INL n.

T: Set of sewer outlet manholes.

D: Set of candidate pipe diameters (24”, 30”, 36”, 42”, … 84”).

Parameters

C_D: The cost of commercial diameter per unit length.

C_{n,m_n}: The costs of manhole m_n associated with depth to upstream crown elevation.

$SE.us_{n,m_n,m_{n+1}}$: Surface elevation at upstream manhole m_n on INL n to manhole m_{n+1} and INL n+1.

$SE.ds_{n,m_n,m_{n+1}}$: Surface elevation at downstream manhole m_n on INL n to manhole m_{n+1} on INL n+1.

$SE.ds_T$: Surface elevation at downstream manhole at outlet manhole T.

$\mathrm{CE.}\,\mathrm{ds_T}$: The crown elevation of the existing pipe at outlet manhole T.

ET: The crown elevation of the existing pipe at outlet manhole T.

$\mathrm{L_{n,m_n,\,m_{n+1},D}}$: Length pipe of the diameter D connecting manhole $\mathrm{m_n}$ on INL n to manhole $\mathrm{m_{n+1}}$ on INL n+1.

$\mathrm{Q_{n,m_n,m_{n+1},}}$: Flow through pipes between manhole $\mathrm{m_n}$ on INL n to manhole $\mathrm{m_{n+1}}$ and INL n+1.

$\mathrm{V_{max}, V_{min}}$: Min-max allowable velocity (3 ft/sec and 15 ft/sec).

$\mathrm{m_D}$: Constant value 2.16 for U.S. units (3.21 for SI units).

D: Pipeline diameters in inches.

BigM: Plenty value = 10^{10}.

Variables

$\mathrm{L_{n,m_n,\,m_{n+1},D}}$: Length of pipe of diameter D connecting manhole $\mathrm{m_n}$ on INL n to manhole $\mathrm{m_{n+1}}$ on INL n+1.

$\mathrm{C\,E.\,us_{n,m_n,m_{n+1}}}$: The depth to upstream crown elevation for manhole $\mathrm{m_n}$ on INL n to manhole $\mathrm{m_{n+1}}$ on INL n+1.

$\mathrm{C\,E.\,ds_{n,m_n,m_{n+1}}}$: The depth to downstream crown elevation for manhole $\mathrm{m_n}$ on INL n to manhole $\mathrm{m_{n+1}}$ on INL n+1.

$\mathrm{CE.}\,ds_T$: The depth to downstream crown elevation manhole at outlet manhole T.

10 RESULTS & CONCLUSIONS

10.1 Summary and Results

The availability of fresh water is limited and has declined in arid and semi-arid areas. Urbanization, agricultural and industrial activities, and climate change have resulted in water stress. However, reused water is considered to be an alternative source that can be used for agricultural and industrial activities to minimize the gap of water supply and demand in those regions. Evaluating strategic plans can play an essential role in overcoming the problems associated with storm and wastewater system planning. The goal of this research was to develop optimization models for planning and designing collection type systems, such as regional wastewater systems and storm water systems. The first model (M-1) was developed to plan a regional wastewater system, considering minimum costs of location, type, and size sewer network and WWTPs. The second model (M-2) was developed to design a regional wastewater system, considering minimum hydraulic design costs, such as pump stations, commercial diameters, excavation costs, and WWTPs. Both models were applied to the Jizan region, KSA. The third model (M-3) was developed to solve the layout and pipe design for storm water systems simultaneously. The model was applied to four different case scenarios, considering two approaches to commercial diameters. The fourth model (M-4) was developed to solve the optimum pipe design of the storm sewer system for a given layout. M-4 was applied to a storm sewer network previously published in the literature. M-1, M-2, and M-3 were developed in the GAMS program and formulated as a MINLP problem, while M-4 was formulated as an NLP. The models were developed using the INL method, which was introduced in the early 1970s. During that time, there were restrictions on applying the concept, due to computer computation space, solution methods, and mathematical formulation.

Now, with the expansion in optimization tools and solvers, many water problems can be solved using these concepts. Chapters 4 and 5 explained in detail the development of the INL method. Chapters 6 and 7 planned and designed a regional wastewater system in the Jizan region, KSA, using the models developed herein. Chapters 8 and 9 considered the layouts and pipe designs for storm sewer systems.

Advantages of these models in regional wastewater systems include:

1) An optimal solution for planning a regional wastewater system.
2) An optimal solution for designing a regional wastewater system.
3) Introducing commercial diameters for pipe design.
4) Introducing pump stations as design constraints.
5) Excavation costs introduced in an objective function.
6) Minimum and maximum velocities are added to hydraulic constraints.

Advantages of these models in storm water systems include:

1) Simultaneous optimal layout and pipe design for a storm sewer system.
2) Optimal design storm sewer system for a given layout.
3) Commercial diameters as design constraints.
4) Minimum and maximum velocities and/or pipe slopes as design constraints.
5) Crown elevations as design constraints for a storm water system.
6) Any form of cost functions included. It can be easily integrated into the objective function for any new or updated cost equation.
7) Computational time for models was quite fast. Running time was estimated to be less than 10-20 seconds for each model.

10.2 Comments on These Models

The optimum solutions for these models were developed in GAMS using MINLP and NLP solvers. In recent years, many solvers introduced in GAMS have been able to solve many water resource problems. Some comments on these models (Mays, 1976) are given below:

1) The simulated annealing (SA) approach was used to design the storm sewer system, which saved over 7% in comparison to the conventional approach. However, the comparison of NLP and SA approaches for the design of a storm sewer system by Karovic and Mays (2014) presented in Chapter 9 shows that NLP gives good results compared to the SA approach.
2) For the layout and pipe design of a storm sewer system, M-3, the connectivity of each connection pipe only solves for the next downstream pipe, which was considered in connectivity formulation model (as described in Chapter 4).
3) The construction of INLs eliminated a large number of possible network configurations in any drainage basin for a system of manholes. However, due to topography, street patterns, hydraulic design constraints, and possible outlet locations, the chances of the INL construction eliminating the optimal layout are rather small (as described in Chapter 5).
4) It may be argued that global optimum solutions may not be obtained due to the approach used to describe the hydraulic of flows. Including a hydraulic or hydrologic routing technique to account for the time lag and attenuation impact of design flows may end up in cheaper pipe designs, especially within the downstream portion of a sewage system. Not accounting for potential flood damage costs within the optimization procedure might prevent optimal

results. For M-1 and M-2, considering reused water after treatment would affect planning and designing systems, which also prevent optimal global results.

10.3 Conclusions

M-1, M-2, and M-4 were developed in this dissertation and applied to case studies. M-3 was applied to a hypothetical system with mild ground slopes and to an actual system with steep ground slopes. Through the use of these scenarios, the following conclusions can be made:

1. An alternate optimal solution can be found by considering different possible outlet locations nodes (WWTP, drainage area, or existing sewer system) and possible connections between manholes. From these different layouts, the model can choose the cheapest or best layout and design.
2. The construction of INLs is not limited by ground surface elevations when using the proposed models, but is restricted by street patterns (flow direction) and locations at the outlets. There are only a few possible drainage line constructions for a given set of manholes located in the INLs for a given topography of the street and outlet location.
3. When ground surfaces are slightly sloped (flat area), pipe slopes tend to be steeper, taking into account the trade-off between the cost of the pipe size and the cost of excavation, resulting in the effect of the pipe's minimum velocity constraints. As the ground surface slopes become steeper, the pipe slopes tend to be parallel with the ground surface, providing the pipe with the effect of maximum velocity limitations.

4. As described in Chapters 5 and 8, cheaper costs will be found in layouts within a lower peak flow rate in the downstream sewer. Therefore, the model should select more pipes in the final manhole to be connected to the outlet, which will result in the layout having slower flow rates and consequently a smaller pipe size. This also means, if possible, selecting more than one outlet.
5. The location and number of possible outlets and number of drainage line constructions may be limited by topographic and physiographic conditions.
6. Dummy nodes for undirected sewer pipes should be introduced. For example, if there is no possible connection between manhole X on INL I and manhole Y on INL I+1, but there is possible connection to manhole Z on INL I+2, here a dummy node can be introduced on INL I+1 to connect manhole X on INL I to manhole Z on INL I+2, which is described in detail in Chapter 6.

10.4 Recommendation for Future Work

In order to make these models more powerful and useful for engineering practices, there are many recommendations to extend them. The following are initial ideas for future research.

10.4.1 Risk and Uncertainty Analysis

Extension of models should include a flow analysis of risk and uncertainty, as proposed by Tang, Mays, and Yen (1975). For example, the optimal layout and pipe design for the sewer system is based on risk analysis. The aforementioned model would be based on minimizing costs so that the cost of installing the sewer system and the cost of potential flood damage during the system's expected service period are balanced.

The basic idea of the described project is to use an optimization model to define layout and design as a system for different uncertain variables. As the name suggests, uncertainty analysis is a technique where analyses are derived quantitatively, based on "uncertainties" in experimentally measured quantities used in the specific form of mathematical relationship to calculate the derived quantity. Different methods could be applied to uncertainties in the optimization model, such as a chance-constrained model. Chance-constrained models have been used widely in water resources problems (Khatavkar & Mays, 2017). The model minimizes total costs by considering uncertainty in such a way that treated waste does not exceed the treatment capacity, with a certain percentage of reliability. However, if the wastewater produced (QR_i) is greater thanx expected, facility capacities may not be satisfied. A chance-constrained model can be formed by replacing the original constraints with probabilistic statements. There are three cases where random elements could occur: (1) only RHS coefficients, b_i, are random, (2) only elements a_{ij} on the LHS of constraints are random, or (3) both elements a_{ij} and b_i are random. The mathematical model of chance constraints can be described as below:

$$z = Max(\sum_{j=1}^{x} c_j x_j) \tag{10.1}$$

Subject to:

$$P\left(\sum_{j=1}^{x} a_{ij} x_j \leq b_i\right) \geq \alpha_i for\ i = 1 \dots m \tag{10.2}$$

$$x_j \geq 0\ for\ j = 1 \dots n$$

As noted above, the statement of probability can be replaced by a linear constraint, replacing the RHS with a constraint evaluated by the function of inverse probability. A different form is needed for a “>=“ constraint. For an equality constraint, there is no chance that any solution will be satisfied with the constraint. The risk is therefore always 100% for a constraint on equality for a continuous distribution of probabilities (www.me.utexas.edu). On the other hand, an uncertainty analysis is a methodology that can be used to calculate general statistical inferences, such as the mean or standard deviation of the gathered data (population).

A sensitivity analysis approach searches for the optimal layout-design of the sewer networks based on best location, type, and size for various input parameters. For example, the population and people’s behavior are uncertain in such a way that the layout/design of the system will change when one of these parameters is modified. The main argument here is that, if the wastewater produced from sources is greater than expected, the wastewater plant capacities may not be satisfied, which could change the optimum solution.

The system layout will vary by the frequency of selecting the facility for each scenario. For the layout, frequency numbers will define the best location for proposed scenarios. My supposition is that, while the goals set forth in wastewater system planning at the regional level are promising in the selected current situation, they may fail to achieve an optimum solution for the design of a wastewater system, which can, in some cases, be blamed on the population growth rate, water consumption, and other adverse effects of generalizing the uncertainty approach. In future work, a case could be made for two indicators of social impact programming and sustainable development: population growth rate and average water consumption.

Building scenarios for different population growth, water consumption per capita, and water quality should be identified in research. When the population and water consumption increase, costs also increase. The layout changes as well, which makes the problem difficult to predict at the considered level. For example, if source A increases by 4% in population growth rate and source B increases by 0.5%, but the average water consumption in source B is higher than source A, obviously, the answer would be different for both the layout and the design of each system.

However, to be specific, this will derive answers to following questions:

What is the optimum layout/design of wastewater systems if the population increases?

What is the optimum layout/design of wastewater systems if the people reduce their water consumption?

To answer these questions, one must perform a sensitivity analysis method by increasing the population in the model and examining water use to see how these parameters are affected by the layout/design of a wastewater system. The costs of the pipeline and wastewater plants are fixed, which need to be as close to reality for any particular region. Many methods can be used in the proposal, such as a survey method, which is an excellent tool to analyze the results and in optimization models.

10.4.2 Hydraulic and Hydrology Routing

Incorporation of a hydraulic or hydrology routing technique can help adequately account for the time lag and attenuation effect of flood wave design progressing through the storm sewer system. The kinematic wave and Muskingum-Cunge models are two routing techniques that can be used in the models. Such techniques require hydrographic design inputs at each manhole location and, hence, a more sophisticated hydrological analysis. One of the approaches can be considered to be an interface tool of the storm water management modeling SWMM simulator for hydraulic and hydrologic routing techniques with an optimization model for layout and pipe design of the storm water system. The model can also be expanded to include storage of detention basins. The amount of water kept in detention could be optimized at any of the system's collection and/or outlet nodes (manhole, WWTP, etc.). Such a procedure would be incorporated by adding a state variable (storage volume, i.e., retention basin size) and a decision variable (downstream discharge).

10.4.3 Optimization Model for Reuse System

Reusing reclaimed wastewater effluent, which is directly discharged from WWTPs into the environment or the industry sector, has received increasing interest as an alternative and reliable water source in many developing countries, and is already used as an alternative source of water in a wide range of developed countries (Aljanabi et al., 2018). Therefore, the best water reuse projects, in term of economic viability and public acceptance, are those that save part of the available freshwater by substituting it with reclaimed water in irrigation or other types of demand. This action is considered friendly to the environment and is an excellent measure to reduce water pollution. Research has indicated that the majority of countries in the Middle East have experience in the field of treated wastewater reuse (Mizyed, 2013; UNESCO, 2017; UN Water, 2017). However, KSA has not yet entered this field; about one-quarter of the treated wastewater (240 million m^3) was used in KSA to irrigate landscape plants, trees, and grass in municipal parks in several cities, including Riyadh, Al Taif, Jeddah, Dhahran, and Jubail in 2010. Baghdad also disposes more than 1.0 million m^3 of treated wastewater into the Tigris river daily after receiving secondary treatment (Aljanabi et al., 2018). The priority is expected to shift from ongoing primary agricultural use to industrial use, with higher anticipated fiscal revenue, such as in the United States. California and Florida especially use wastewater as an alternative source in many applications, including industrial, domestic, commercial, groundwater recharge, and recreational purposes. Use of treated wastewater is projected to be greatest in Riyadh, which uses more than 100 million m^3 of treated wastewater in agricultural, recreational, ecological, landscaping, industrial, and groundwater recharge purposes. This represented 50% of treated wastewater in 2010. After an advanced treatment process, it will be possible to produce about 2.5 km^3/year of this type of treated wastewater in 2035.

10.4.3.1 Reclaimed Water as an Alternative Source

Because of the increase in urban development, along with a rapid increase in population, reclaimed wastewater, which can be converted into a reliable alternative water source with limited uses, deserves more interest (Aljanabi et al., 2018). It is important to include this water source in future planning and implementation of water resource projects, especially due to its enormous volume in KSA in general. Wastewater reuse in agricultural irrigation is one of the best known and applicable to Middle East countries; other uses, such as environmental restoration, cleaning, toilet flushing, car washing, power plant cooling systems, air conditioning, groundwater recharge, and industrial applications, are also applicable. All of these uses are practiced today in most arid and semi-arid regions around the world, especially in Mediterranean countries, which are facing a significant challenge due to an increase in water shortages. Agricultural reclaimed water reuse is a common practice in several Mediterranean countries and the long-term effect of treated wastewater on cultivated crops or human consumption and other related uses is of considerable interest. Expansion of the model will include reuse optimization models, as proposed by Aljanabi et al. (2018a). For example, optimal water reuse planning would consider the hydraulic design of the system, with allocations of treated wastewater to different users. Such a model would be based on minimizing costs so that a proper balance is maintained between the costs of installing a sewer system and the profit of using treated wastewater by different users and applications.

10.4.4 Sustainability of Wastewater/Storm Water Systems

Due to a significant increase in population with limited access to freshwater resources, considerable water stress is a reality for a substantial portion of the world. According to UNESCO (2017),

> In the face of ever-growing demand, wastewater is gaining momentum as a reliable alternative source of water, shifting the paradigm of wastewater management from 'treatment and disposal' to 'reuse, recycle and resource recovery.'

Mays (2007) presented the following definition of water resource sustainability:

> Water resources sustainability is the ability to use water in sufficient quantities and quality from the local to the global scale to meet the needs of humans and ecosystems for the present and the future to sustain life and to protect humans from the dangers brought about by natural and human-caused disasters that affect sustaining life.

One of the sustainable development goals of the UN Development Program is clean water and sanitation to satisfy real improvement in water use efficiency, protecting the environment, maintaining available water resources, and any other action to improve water use. Structural solutions are often more expensive and can result in more significant environmental damage than a nonstructural solution. Non-structural measures help reduce financial pressure and ecological disaster.

Traditional ways that have been used in urban water management have aimed to meet supply and demand by conveying wastewater or storm water from the city or urban places. According to Foxon et al. (2002), there are four categories of sustainability criteria of water/waste system asset development decisions. These categories consider the economic, environmental, and social principles of sustainability, together with technical standards, which relate primarily to the ability of the water/wastewater system to sustain and enhance the performance of the functions for which it is designed. Table 10.1 shows a set of primary criteria. Assessment of sustainability was based on four criteria of urban water management systems: environmental, economic, social, and technical (Makropoulos et al., 2008). The primary consideration was how to integrate sustainability criteria and apply them to the optimization model.

Table 10.1. Primary Criteria for Sustainability Assessment (Foxon et al., 2002).

Category	Primary criterion
Economic	Life cycle costs
	Willingness to pay
	Affordability
	Financial risk exposure
Environmental	Resource utilization
	Service provision
	Environmental impact
Social	Impact on risks to human health
	Acceptability to stakeholders
	Participation and responsibility
	Public awareness and understanding
	Social inclusion
Technical	Performance of the system
	Reliability
	Durability
	Flexibility and adaptability

10.4.4.1 Economic Criteria

Economic indicators represent the costs associated with the construction and operation of WWTPs, pump stations, and pipelines. They are often decisive when choosing a technology in a practical situation (Balkema et al., 2002). The two most commonly used indicators are cost of investment and cost of operation and maintenance. It should be noted that, when the investment costs of different WWTP technologies are compared, it is essential to consider the lifespan of each technology. It also represents the costs associated with the construction of pipeline diameters, including different prices for each diameter, length, and type of pipeline. Moreover, it represents the costs associated with the construction and operation of pumps, and there are different types of pump operations with different costs. Lifecycle costs of each scenario (i.e., capital cost, end-user cost, remediation costs, etc.), affordability for the public, and financial risks (Foxon et al., 2002) are considered to be economic objectives.

The costs and efficiency for each WWTP technology are various; Hernandez-Sancho et al., 2011) applied the optimization model and looked for technology that resulted in the optimum solution. There are different types of technologies in such way of costs, total capacities, and efficiencies removal, like SS, COD, and BOD. The costs of the diameter of a pipeline depend on the type of pipeline (material) and construction. Karovic and Mays (2014) summarized the total cost for constructing a linear foot of storm sewer pipe. The cost includes the price of the pipe, pavement removal for trench excavation, trench excavation, and final backfill. Brand and Ostfeld (2011) developed construction cost equations for regional wastewater system components according to the results of tenders for governmental offices, authorities, and private bodies in Israel. The database includes prices of sections in the fields of civil engineering, construction, concrete, installations, electricity, and various construction materials. Swamee and Sharma (2007) presented different cost functions for different pipe materials.

10.4.4.2 Environmental Criteria

Environmental sustainability presents the ability of the natural world to withstand the impact of human activity (van der Vleuten-Balkema, 2003). Wastewater carries a lot of components that can affect nature and people. Water quality constraints can be applied to different required standards and the efficiency of WWTP for various chemical elements. Cornejo et al. (2016) used a life cycle assessment to evaluate the scale of implementation’s impact on the environmental sustainability of wastewater treatment, considering water reuse, energy recovery, and nutrient recycling. Environmental sustainability criteria are offered for wastewater system infrastructure in terms of wastewater quality constraints (i.e., pollutant concentration limits and treatment efficiencies of WWTP). Proposed alternatives include water quality standards for different usages and/or using efficiency as a decision variable that can be identified as environmental criteria. The efficiencies of treatment plants and standard requirements that should be satisfied before water leaves treatment plants should be considered. Moreover, regarding blending wastewater at collection and wastewater treatment efficiency nodes, the water quality would be changed, which must be taken into account (Mays et al., 1983).

10.4.4.2.1 Water Quality

The importance of water is widely recognized, and the need to maintain its quality has led to the introduction of many environmental guidelines and regulations to limit pollutant discharge into water bodies. According to the UN Water Annual International Zaragoza Conference (2015), by 2030, water quality could be improved by reducing pollution, eliminating dumping, minimizing the disposal of dangerous chemicals and materials, halving the percentage of untreated wastewater, and improving global recycling and securing reuse. In the European Union, the introduction of the Water Framework Directive (Directive 2000/60/CE) offered an integrated vision of water resources with the aim of achieving “good water status” for all water bodies. For instance, rather than just imposing standards for pollutant discharge, water quality standards are explicitly defined for receiving water bodies through a river basin-scale approach. With the same goal of water sustainability, holistic methods for water resources have been progressively applied in other developed countries, prompting similar water quality standards (e.g., National Recommended Water Quality Criteria in the United States and the National Water Quality Management Strategy in Australia).

The problem associated with designing wastewater systems at the regional level is providing water to remote communities while meeting water quality and quantity requirements, which involve intensive treatment, and use it as another supply source. Many alternative solutions can be added to the proposed models as environmental constraints, such as wastewater quality standards and use efficiency of WWTP as a decision variable for quality purposes. In this dissertation, due to the problems of a decreasing rural population and an increasing urban population, the future population, water consumption, and water quality scenarios were addressed, suggesting explicitly shifting from a centralized to a decentralized wastewater system; optimization models associated with populations or distances were investigated. Further, water blends at collection and wastewater treatment nodes; therefore, water quality changes at collection and wastewater treatment nodes and must be taken into account.

10.4.4.3 Sustainability Index (SI)

Several indexes are related to standards of sustainability and include the reliability, resilience, and vulnerability of the water supply system, environmental system integrity through consideration of water quantity and quality, spatial and temporal equity, and socio-economic acceptability (Oxley et al., 2016)). Aydin et al. (2014), Loucks (1997), and Sandoval-Solis et al. (2011) used the concept of a sustainability index (SI) to measure the sustainability of water systems. The family of water resources systems, such as water supply management, water distribution systems, and groundwater management, has been connected to sustainability in many previous applications.

The expansion of SI includes resiliency, reliability, and vulnerability performance criteria. Evolution of water resources sustainability can be achieved using optimization approaches. Considering the case study in this dissertation, one approach could be a multi-objective genetic algorithm with the following objectives:

Objective 1: Minimize total installation and operation costs and maintain the cost of the design of the storm/wastewater system.

Objective 2: Maximize the SI of the entire system.

Another way to evaluate the design storm/wastewater collection's sustainability using optimization techniques would be based on a single objective of either maximizing the SI or minimizing the cost of the entire system. The following objectives are proposed for this purpose:

Example 1:

Objective: Minimize total installation and operation costs and maintain the cost of the design of the storm/wastewater system.

Subject to: SI of the entire system for flow deficits at sewer systems, WWTP (outlet node), and water quality (concentrations of treated wastewater; outlet node).

Example 2:

Objective: Maximize SI for flow deficits at sewer systems and WWTP (outlet node) and water quality (concentrations leaving WWTPs; outlet node).

Subject to: Total installation and operation costs and maintaining the cost of the design of the storm/wastewater system.

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APPENDIX A

STATISTICS INFORMATION REGARDING THE WATER SITUATION IN KSA FROM THE MINISTRY OF ENVIRONMENT WATER AND AGRICULTURE (MEWA)

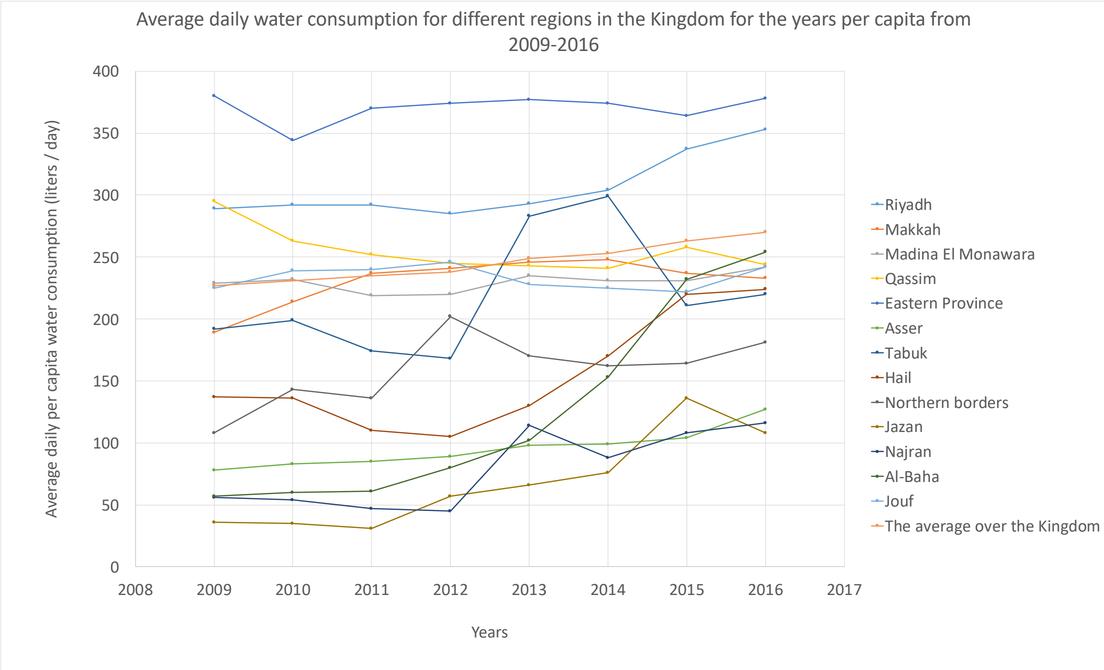

Figure A.1. Average Daily Water Consumption for Each Region in the Kingdom Per Capita From 2009-2016 (MEWA, 2017).

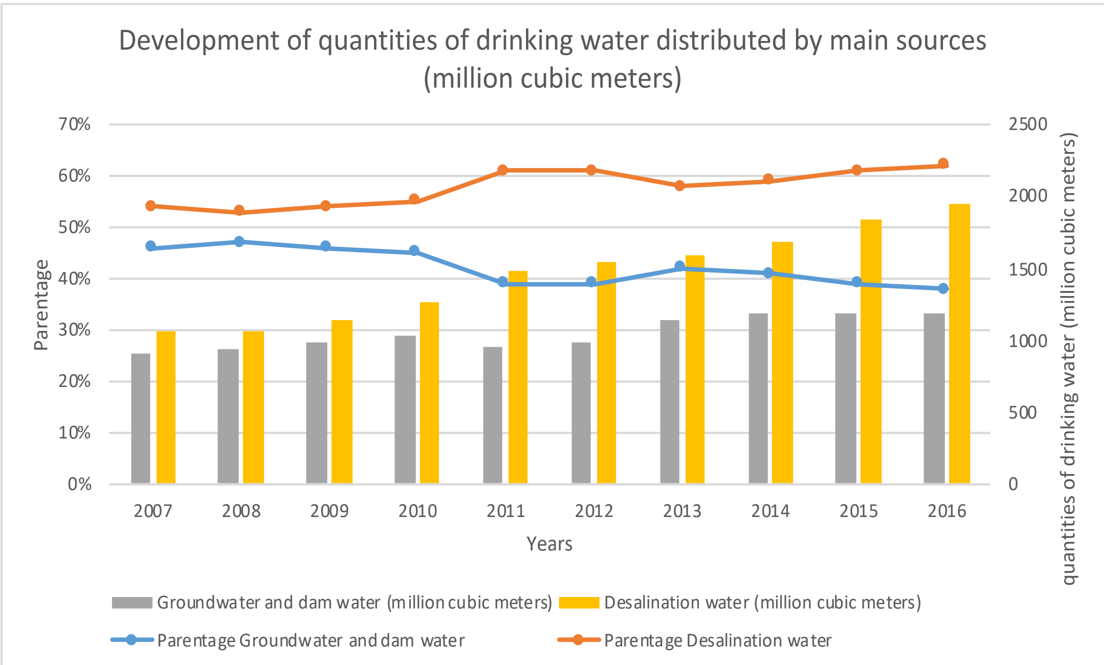

Figure A.2. Fluctuation of Quantities of Drinking Water Distributed by Main Sources (MEWA, 2017).

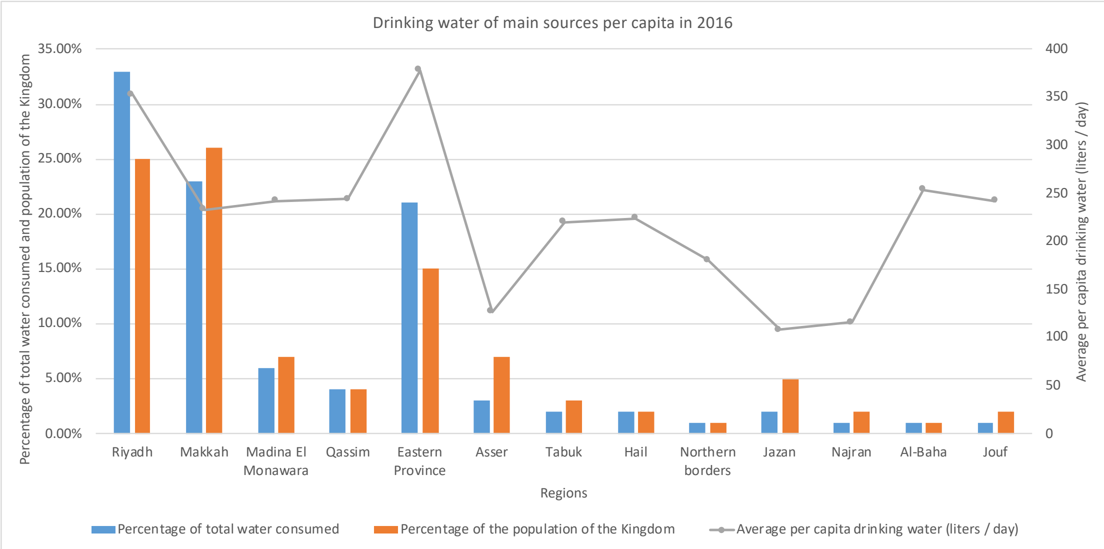

Figure A.3. Average Daily Water Consumption for Each Region per Capita in 2016 (MEWA, 2017).

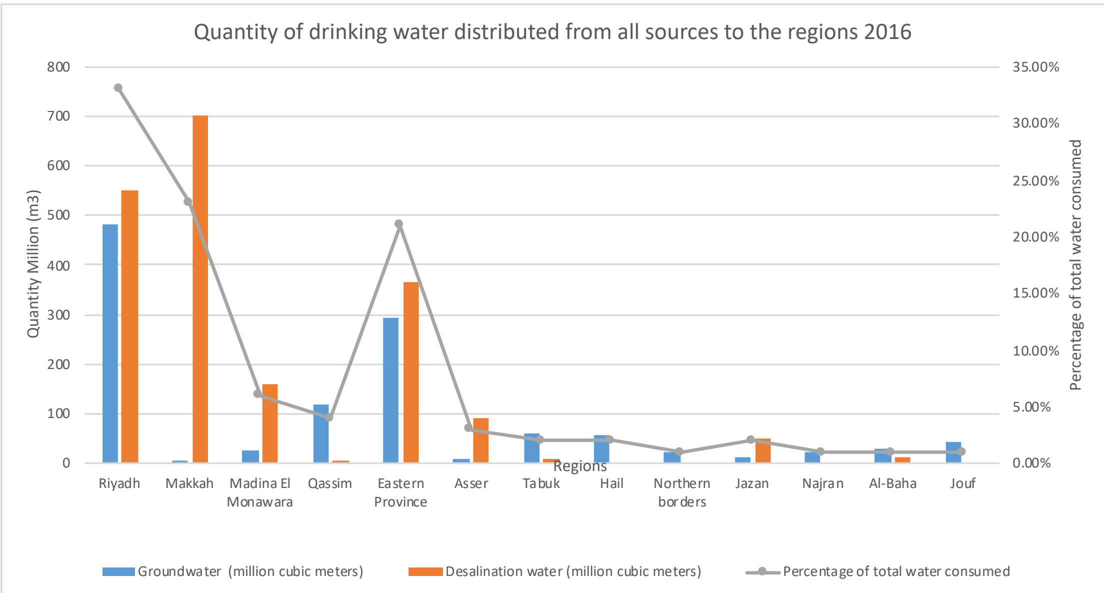

Figure A.4. Quantity of Drinking Water Distributed from All Sources to the Regions in 2016 (MEWA, 2017).

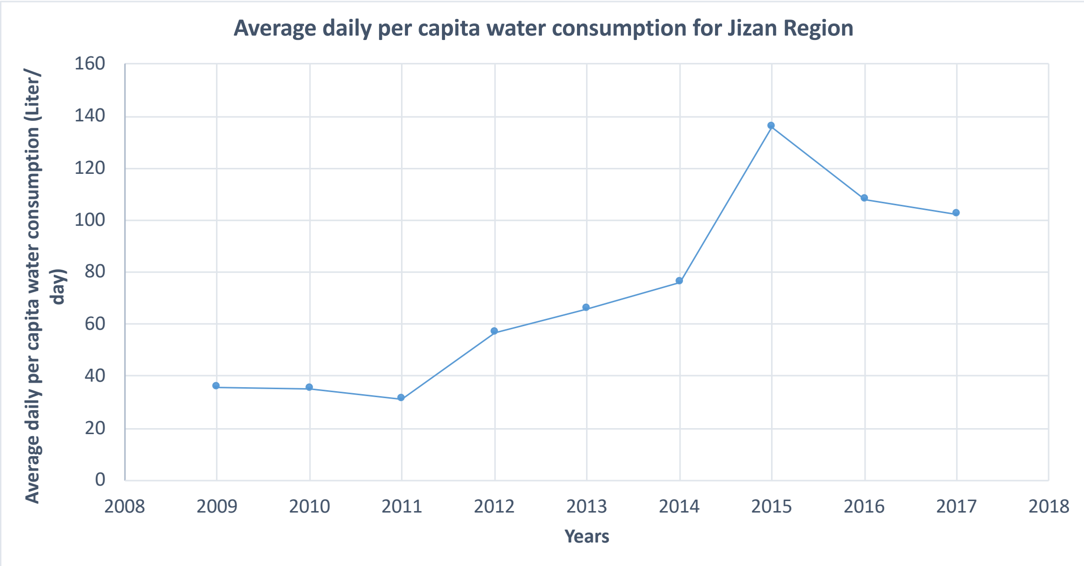

Figure A.5. Average Daily Water Consumption for Jizan Region per Capita From 2009-2017 (MEWA, 2017).

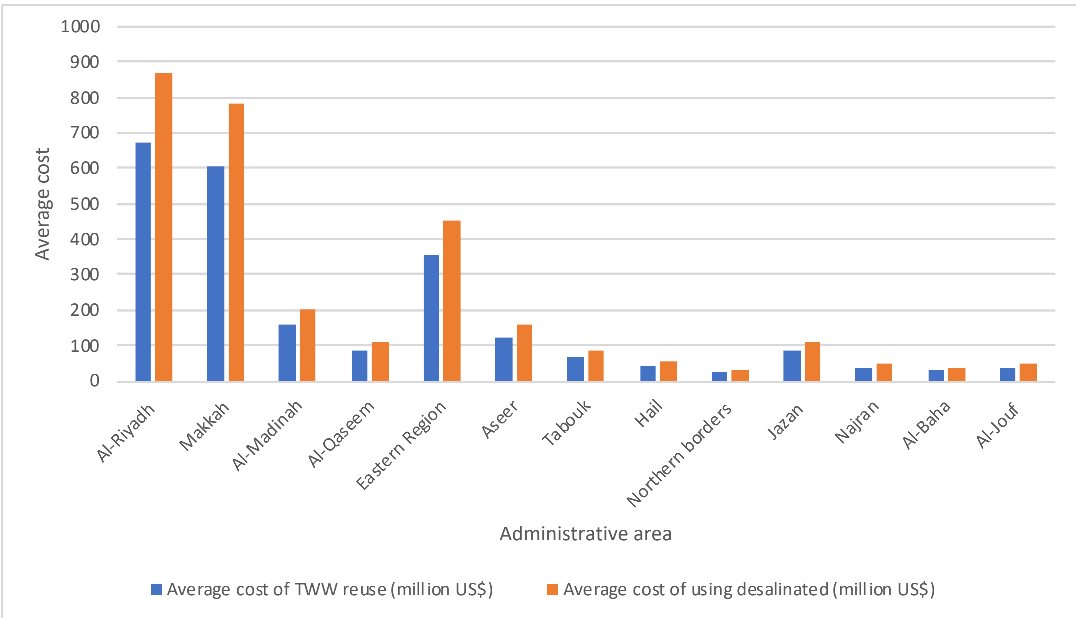

Figure A.6. Cost Comparison of Using Treated Wastewater (TWW) and Desalinated Water (adapted from Chowdhury & Al-Zahrani, 2013).

APPENDIX B

CALCULATIONS OF WASTEWATER GENERATED IN EACH CITY

Table B. 1. Future Population in Jizan Region, Saudi Arabia. The General Authority for Statistics (Under title: Directory of Services 2017, Jizan region, “In Arabic”).

No.	Cities	Population 2017	Population 2020	Population 2025	Population 2030
1	Jazan	127743	136050	148862	160456
2	Al Fatiha	7021	7366	7861	8264
3	Sabya	165967	176759	193405	208469
4	Alaliya	25744	27583	30484	33188
5	Qawz al Ja’afirah	18794	20016	21901	23607
6	Al Kadami	17870	18749	20008	21034
7	Abu Arish	144667	154074	168584	181714
8	Wadi Jizan	52445	55024	58719	61730
9	Samtah	101033	105367	111324	115867
10	AlGofol	24817	26037	27786	29211
11	Al Sehi	21480	22536	24050	25283
12	Al Khubah/Alharth	8342	8752	9340	9819
13	Khushal	10244	10748	11470	12058
14	Damad	52193	54759	58437	61433
15	Al Shugayri	19408	20546	22258	23752
16	Al Darb	36583	39196	43319	47162
17	Al Qasabah	18281	19180	20468	21517
18	Al Reeth	14296	14999	16006	16827
19	Baish	37108	38700	40888	42556
20	Al Haqu	10259	10763	11486	12075
21	Masliyah	17359	18377	19908	21245
22	Alaydabi	7672	8049	8590	9030
23	Addayer	34336	37909	44044	50410
24	Ahad Al Masarihah	85965	90192	96249	101184
25	Al Madaya/Al-Hakamih	24745	26196	28378	30284
26	Al Aridhah	45946	48205	51443	54080
27	Alhumira	9879	10365	11061	11628
28	Al Shuqaiq	23875	25049	26731	28102
29	Al Tuwal	36547	38691	41913	44728
30	Harub	15934	16717	17840	18755
31	Fayfa	29793	31258	33357	35067
32	Itwide	5081	5331	5689	5981
33	Aiban/Belghazi	24063	25246	26942	28323
34	Al Mwassam	17779	18653	19906	20927

Table B.2. Future Population Growth in Jizan Region, Saudi Arabia (MEWA, 2017).

No.	Cities	Population growth 2020	Population growth 2025	Population growth 2030
1	Jazan	2.1	1.8	1.5
2	Al Fatiha	1.6	1.3	1
3	Sabya	2.1	1.8	1.5
4	Alaliya	2.3	2.0	1.7
5	Qawz al Ja’afirah	2.1	1.8	1.5
6	Al Kadami	1.6	1.3	1
7	Abu Arish	2.1	1.8	1.5
8	Wadi Jizan	1.6	1.3	1
9	Samtah	1.4	1.1	0.8
10	AlGofol	1.6	1.3	1
11	Al Sehi	1.6	1.3	1
12	Al Khubah/Alharth	1.6	1.3	1
13	Khushal	1.6	1.3	1
14	Damad	1.6	1.3	1
15	Al Shugayri	1.9	1.6	1.3
16	Al Darb	2.3	2.0	1.7
17	Al Qasabah	1.6	1.3	1
18	Al Reeth	1.6	1.3	1
19	Baish	1.4	1.1	0.8
20	Al Haqu	1.6	1.3	1
21	Masliyah	1.9	1.6	1.3
22	Alaydabi	1.6	1.3	1
23	Addayer	3.3	3.0	2.7
24	Ahad Al Masarihah	1.6	1.3	1
25	Al Madaya/Al-Hakamih	1.9	1.6	1.3
26	Al Aridhah	1.6	1.3	1
27	Alhumira	1.6	1.3	1
28	Al Shuqaiq	1.6	1.3	1
29	Al Tuwal	1.9	1.6	1.3
30	Harub	1.6	1.3	1
31	Fayfa	1.6	1.3	1
32	Itwide	1.6	1.3	1
33	Aiban/Belghazi	1.6	1.3	1
34	Al Mwassam	1.6	1.3	1

The generated wastewater is calculated based on leakage factor 20%, water demand of 0.25 (m^3/person/day) for 85,000 people and 0.2 (m^3/person/day) for less 85,000 people (MEWA, 2016), wastewater network coverage 100% for each city, and factor safety of 2.

Table B. 3. The Generated Wastewater for Each City, Jizan Region, Saudi Arabia (MEWA, 2017).

No.	Cities	Wastewater (m^3/day)	No.	Cities	Wastewater (m^3/day)
1	Jazan	25673	18	Al Reeth	2154
2	Al Fatiha	1058	19	Baish	5447
3	Sabya	33355	20	Al Haqu	1546
4	Alaliya	4248	21	Masliyah	2719
5	Qawz al Ja'afirah	3022	22	Alaydabi	1156
6	Al Kadami	2692	23	Addayer	6453
7	Abu Arish	29074	24	Ahad Al Masarihah	16189
8	Wadi Jizan	7901	25	Al Madaya/ Al-Hakamih	3876
9	Samtah	18539	26	Al Aridhah	6922
10	AlGofol	3739	27	Alhumira	1488
11	Al Sehi	3236	28	Al Shuqaiq	3597
12	Al Khubah/ Alharth	1257	29	Al Tuwal	5725
13	Khushal	1543	30	Harub	2401
14	Damad	7863	31	Fayfa	4489
15	Al Shugayri	3040	32	Itwide	766
16	Al Darb	6037	33	Aiban/Belghazi	3625
17	Al Qasabah	2754	34	Al Mwassam	2679

APPENDIX C

GAMS CODE USED TO SOLVE THE EXAMPLE APPLICATION OF AN OPTIMIZATION MODEL FOR PLANNING REGIONAL WASTEWATER SYSTEMS, DEVELOPED IN CHAPTER 4, M-1

```
*THIS IS MODEL FOR SITTING AND SIZING THE COMPONENTS OF A REGIONAL WASTEWATER
*SYSTEM (THE ESSENTIAL COMPONENTS ARE SWERE NETWORKS AND WASTEWATER
TREATMENT PLANTS)
*MIXED INTEGER NON-LINEAR PROGRAMMING (MINLP) IS USED TO MINIMIZE TOTAL COSTS.
*MODELED BY FAISAL ALFAISAL
*ADVISER. PROF.LARRY MAYS
*WED-02 MAY


Sets
   i   Sources Nodes                        /n1, n2, n3/
   j   Candidate Collection Nodes           /n4, n5, n6/
   k   Candidate WWTP Nodes                 /n7, n8, n9/
     ;

Parameters

   QR(i) Wastewater produced at source nodes i which represent supply nodes (Mgal/day)
    /  n1     18
       n2     46
       n3     74 /


* Zero values mean that flow coming to collection node should leave with same amount

   QI(j) Transshipment at candidate collection nodes j which represent collection nodes
    /   n4    0
        n5    0
        n6    0
              /
 ;

*This value is greater than total wastewater produced at source node I so there is no capacity limitation applied
*for this model.

scalar    QMAX      Treatment Capacity /150/ ;


* Assumed total costs for pipes that connecting from source nodes i to candidate collection nodes j

Table CostPip(i,j)  total costs  from nodes i to j  (M$ PER FLOW)
              n4          n5        n6
   n1        1.5         1.5       1.2
   n2        1.5         1.2       1.2
   n3        1.5         12        1.2   ;


* Assumed total costs for pipe that connecting from candidate collection nodes j to candidate WWTP nodes k

Table CostPipe(j,k)  total  cost  from nodes j to k  (M$ PER FLOW)
             n7      n8      n9
   n4       1.2     1.8      1.8
   n5       10      1.8      1.8
   n6       1.0     1.8      1.8
      ;


*These are the decision variables
Variables


   COST      The optimum cost;
```

```
Positive Variable
            QS(i,j)                    Wastewater rate in sewer system from source i to candidate collection node j
             QC(j,k)             Wastewater rate in sewer system from candidate collection j to candidate WWTP
node k
             QT                  Wastewater treated at candidate WWTP k

             ;

* Variable binary that will take value 1 if there is a sewer linking node i to node j and 0 otherwise
Binary Variable x(i,j)  ;

* Variable binary that will take value 1 if there is a sewer linking node j to node k and 0 otherwise
Binary Variable y(j,k)   ;


Equations
    cost1           Objective function
    Source          Continuity constraint at nodes i
    Collection      Continuity constraint at nodes j
    WWTP            Continuity constraint at nodes k
    TCapacity       Treatment plant capacity
    Massbal         Total wastewater that produced should be treated
    Conduct1        Conductivity constraint from source nodes i to candidate collection nodes j
    Conduct2        Conductivity constraint from candidate collection nodes j to candidate WWTP nodes k

* Upper and lower bound
    maxUpp          Upper Bounded Eq for maximum amount of flow allowed  on arc i and j
    maxUpp1         Upper Bounded Eq for maximum amount of flow allowed  on arc j and k
    minLo           Lower Bounded Eq for minimum amount of flow allowed  on arc i and j
    minLo1          Lower Bounded Eq for minimum amount of flow allowed  on arc j and k
  ;

*The produce flow QR(i) at source nodes i should be equal to the sum of the conveyed flow QS(i,j)
* from source nodes i to candidate collection nodes j,

Source(i)..              QR(i) =E=sum(j, QS(i,j)*x(i,j)) ;


* The difference between the sum of the total collected inflow QS(i,j) at candidate collection nodes j
* minus the sum of the total outflow to candidate wastewater treatment plants k have to be equal zero

Collection(j)..          sum(i, QS(i,j)*x(i,j))-sum(k, QC(j,k)*y(j,k)) =e= QI(j) ;

* The sum of the conveyed flow QC(j,k) from candidate collection nodes j to candidate WWTP nodes k, have to
be * equal to outflow (treated wastewater) at each candidate WWTP node k.


WWTP(k)..                sum(j, QC(j,k)*y(j,k)) =e=QT(k);


* The sum of the total produced wastewater QR(i) at source nodes i should be equal to the sum of
* wastewater treated QT(k) at candidate WWTP nodes k

Massbal..                sum(i, QR(i))=E=sum(k, QT(k));

* The sum of the conveyed flow QC(j,k) from candidate collection nodes j to candidate WWTP nodes k, have to
be * equal or less than maximum WWTP capacity QMAX, this constraint can be applied if there is existing
WWTP with * knowing total capacity.

TCapacity(k)..           QMAX=G=sum(j, QC(j,k)*y(j,k));

* The flow from each source node i must flow through one candidate collection nodes j
Conduct1(i)..            sum(j, x(i,j))=e=1 ;
```

```
* The flow from each candidate collection nodes j must flow through one candidate WWTP nodes k if there is
flow * QC(j,k)

Conduct2(j)..              sum(k, y(j,k))=e= 1*(sum(k, QC(j,k)))/(ABS((sum(k, QC(j,k))-1))+1) ;

*---------------------------------------------------------------------------
* Upper and lower bound for the flow through the sewer system have to be between maximum and
* minimum values for example : 74 Mgal/d is upper value for the flow in sewer from source node i to candidate
* collection nodes j

maxUpp(i,j)..         QS(i,j) =l= 74*x(i,j);
maxUpp1(j,k)..        Qc(j,k) =l= 138*y(j,k);
minLo(i,j)..          QS(i,j) =g= 0*x(i,j);
minLo1(j,k)..         Qc(j,k) =g= 0*y(j,k);
*---------------------------------------------------------------------------


* objective function which is a function of flow

cost1..      COST =e= sum((i,j),( CostPip(i,j)*QS(i,j)*x(i,j)))+sum((j,k), CostPipe(j,k)*QC(j,k)*y(j,k))+
SUM(k,3.2*QT(k)) ;


*-------------------------------------------------------------------------
MODEL examfaisal /ALL/;
*-------------------------------------------------------------------------
SOLVE examfaisal USING MINLP MINIMIZING COST;

*Here the model will use BARON solver to solve the problem
OPTION MINLP=BARON;
*-------------------------------------------------------------------------
DISPLAY QS.L,QC.L,QT.L, cost.l, x.l, y.l ;
```

APPENDIX D

GAMS CODE USED TO SOLVE THE APPLICATION OF AN OPTIMIZATION MODEL FOR PLANNING REGIONAL WASTEWATER SYSTEMS, DEVELOPED IN CHAPTER 6 FOR ZONE 3, M-1

```
*THIS IS MODEL FOR ZONE 3 -CASE STUDY- JIZAN REGION SAUDI ARABIA
*MODELED BY FAISAL ALFAISAL
*ADVISER. PROF.LARRY MAYS
*THURSDAY-07 MARCH


Sets
    I1   First iso-nodal Line have one nodes                  /n31/
    I2   Second iso-nodal line have three nodes           /n7,n30/
    I3   Third iso-nodal line have three nodes            /D_N1,D_N2,n33,D_N3,n14/
    I4   Fourth iso-nodal line have one nodes             /D_N1,D_N2,n24,D_N3,D_N4,n10,D_N5/
    I5   Fifth iso-nodal line have one nodes             /D_N1,D_N2,n11,D_N3,n13,n2/
    I6   Sixth iso-nodal line have one nodes              /D_N1,D_N2,n5,D_N3,D_N4/
    I7   Seventh iso-nodal line have one nodes            /W-7,W-30,W-5,W-13,W-2/
      ;

Parameters

    QN1(I1)  Wastewater produced  of nodes n  on iso-nodal line 1
     /  n31          4488.6
   /

* Zero value = collection node or certain value = source node

    QN2(I2)  Transshipment at nodes N  on iso-nodal line 2
     /  n7       6452.5
        n30      2400.6

                  /
* Zero value = collection node or certain value = source node

    QN3(I3)   Transshipment at nodes N  on iso-nodal line 3
     /      D_N1            0
            D_N2            0
            n33           3625.3
            D_N3            0
            n14           2692.3
  /
* Zero value = collection node or certain value = source node

    QN4(I4)   Transshipment at nodes N  on iso-nodal line 4
     /   D_N1            0
         D_N2            0
         n24          1155.8
         D_N3            0
         D_N4            0
         n10          4248.1
         D_N5            0
           /

* Zero value = collection node or certain value = source node

    QN5(I5)   Transshipment at nodes N  on iso-nodal line 5
     /  D_N1            0
        D_N2            0
        n11          3040.3
        D_N3            0
        n13          3021.6
        n2          33354.9

          /
* Zero value = collection node or certain value = source node

    QN6(I6)   Transshipment at nodes N  on iso-nodal line 6
```

/ D_N1 0
D_N2 0
n5 7863.4
D_N3 0
D_N4 0
/

Table L1(I1,I2) The length required from nodes N on iso-nodal line I1 to nodes N on iso-nodal line I2 in Km

	n7	n30
n31	9	100000

Table L2(I2,I3) The length required from nodes N on iso-nodal line I2 to nodes N on iso-nodal line I3 in Km

	D_N1	D_N2	n33	D_N3	n14
n7	0	1000000	7.7	1000000	1000000
n30	1000000	0	24	23	22

;

Table L3(I3,I4) The length required from nodes N on iso-nodal line I3 to nodes N on iso-nodal line I4 in Km

	D_N1	D_N2	n24	D_N3	D_N4	n10	D_N5
D_N1	0	1000000	1000000	1000000	1000000	1000000	1000000
D_N2	1000000	0	1000000	1000000	1000000	1000000	1000000
n33	1000000	1000000	12.7	1000000	1000000	1000000	1000000
D_N3	1000000	1000000	0	1000000	1000000	1000000	1000000
n14	1000000	1000000	1000000	16	17.5	24	20

;

Table L4(I4,I5) The length required from nodes N on iso-nodal line I4 to nodes N on iso-nodal line I5 in Km

	D_N1	D_N2	n11	D_N3	n13	n2
D_N1	0	1000000	1000000	1000000	1000000	1000000
D_N2	1000000	0	1000000	1000000	1000000	1000000
n24	1000000	1000000	16	1000000	1000000	1000000
D_N3	1000000	1000000	0	1000000	1000000	1000000
D_N4	1000000	1000000	1000000	0	1000000	1000000
n10	1000000	1000000	1000000	1000000	17	16
D_N5	1000000	1000000	1000000	1000000	1000000	0

;

Table L5(I5,I6) The length required from nodes N on iso-nodal line I5 to nodes N on iso-nodal line I6 in Km

	D_N1	D_N2	n5	D_N3	D_N4
D_N1	0	1000000	1000000	1000000	1000000
D_N2	1000000	0	1000000	1000000	1000000
n11	1000000	1000000	6	1000000	1000000
D_N3	1000000	1000000	0	1000000	1000000
n13	1000000	1000000	1000000	0	1000000
n2	1000000	1000000	1000000	1000000	0

;

Table L6(I6,I7) The length required from nodes N on iso-nodal line I6 to nodes N on iso-nodal line I7 in Km

	W-7	W-30	W-5	W-13	W-2
D_N1	0	1000000	1000000	1000000	1000000
D_N2	1000000	0	1000000	1000000	1000000
n5	1000000	1000000	0	1000000	15
D_N3	1000000	1000000	1000000	0	1000000
D_N4	1000000	1000000	1000000	1000000	0

;

scalar COSTLEN Costs of sewer per unit length in (Km) and flow (m3/d) /30/

```
*These are the decision variables

Variables
            COST       The optimum cost

;

Positive Variable
            QS(I1,I2)          Wastewater in sewer system
            QC(I2,I3)          Wastewater in sewer system
            QO(I3,I4)          Wastewater in sewer system
            QF(I4,I5)          Wastewater in sewer system
            QG(I5,I6)          Wastewater in sewer system
            QN(I6,I7)          Wastewater in sewer system
            QT                 Wastewater treated in WWTP

            ;

* Variable binary that will take value 1 if there is a sewer linking node on iso-nodal line I1 to node on iso-nodal
line I2 and 0 otherwise
Binary Variable x(I1,I2)  ;

* Variable binary that will take value 1 if there is a sewer linking node on iso-nodal line I2 to node on iso-nodal
line I3 and 0 otherwise
Binary Variable y(I2,I3)   ;

* Variable binary that will take value 1 if there is a sewer linking node on iso-nodal line I3 to node on iso-nodal
line I4 and 0 otherwise
Binary Variable z(I3,I4)  ;

* Variable binary that will take value 1 if there is a sewer linking node on iso-nodal line I4 to node on iso-nodal
line I5 and 0 otherwise
Binary Variable s(I4,I5)  ;

* Variable binary that will take value 1 if there is a sewer linking node on iso-nodal line I5 to node on iso-nodal
line I6 and 0 otherwise
Binary Variable O(I5,I6)  ;

* Variable binary that will take value 1 if there is a sewer linking node on iso-nodal line I6 to node on iso-nodal
line I7 and 0 otherwise
Binary Variable H(I6,I7)  ;

* Variable binary that will take value 1 if there is a WWTP located on iso-nodal line I7
Binary Variable t(I7)   ;


Equations
    cost1          Objective function
    Flow           Continuity constraint for iso-nodal line I1
    Flow1          Continuity constraint for iso-nodal line I2
    Flow2          Continuity constraint for iso-nodal line I3
    Flow3          Continuity constraint for iso-nodal line I4
    Flow4          Continuity constraint for iso-nodal line I5
    Flow5          Continuity constraint for iso-nodal line I6
    Flow6          Continuity constraint for iso-nodal line I7

    Conduct1       Conductivity constraint from iso-nodal line I1  to iso-nodal line I2
    Conduct2       Conductivity constraint from iso-nodal line I2  to iso-nodal line I3
    Conduct3       Conductivity constraint from iso-nodal line I3 to  iso-nodal line I4
    Conduct4       Conductivity constraint from iso-nodal line I4 to  iso-nodal line I5
    Conduct5       Conductivity constraint from iso-nodal line I5 to  iso-nodal line I6
    Conduct6       Conductivity constraint from iso-nodal line I6 to  iso-nodal line I7

* Upper and lower bound
```

```
   maxUpp        Upper bounded Eq for maximum amount of flow allowed  on arc iso-nodal line I1 and I2
   maxUpp1       Upper bounded Eq for maximum amount of flow allowed  on arc iso-nodal line I2 and I3
   maxUpp2       Upper bounded Eq for maximum amount of flow allowed  on arc iso-nodal line I3 and I4
   maxUpp3       Upper bounded Eq for maximum amount of flow allowed  on arc iso-nodal line I4 and I5
   maxUpp4       Upper bounded Eq for maximum amount of flow allowed  on arc iso-nodal line I5 and I6
   maxUpp5       Upper bounded Eq for maximum amount of flow allowed  on arc iso-nodal line I 6 and I7

 ;

* The produce flow at nodes N on iso-nodal line I1 QN1(I1) should be equal to the sum of the conveyed flow
*QS(I1,I2) from nodes N on  iso-nodal line I1 to nodes N on  iso-nodal line I2

Flow(I1)..          QN1(I1) =e=sum(I2, QS(I1,I2)*x(I1,I2)) ;


* The difference between the sum of total collected inflow QS(I1,I2) at nodes N on iso-nodal line I1 minus
* the sum of the total outflow QC(I2,I3) to nodes N on iso-nodal line I3 have to be equal -QN2(I2)
* (it equal zero for collection node, certain values for source node)

Flow1(I2)..          sum(I1, QS(I1,I2)*x(I1,I2))-sum(I3,QC(I2,I3)*y(I2,I3)) =e= -QN2(I2) ;


* The difference between the sum of total collected inflow QC(I2,I3) at nodes N on iso-nodal line I2 minus
* the sum of the total outflow QO(I3,I4) to nodes N on iso-nodal line I4 have to be equal -QN3(I3)
* (it equal zero for collection node, certain values for source node)

Flow2(I3)..          sum(I2, QC(I2,I3)*y(I2,I3))-sum(I4,QO(I3,I4)*z(I3,I4)) =e= -QN3(I3) ;


* The difference between the sum of total collected inflow QO(I3,I4) at nodes N on iso-nodal line I3 minus
* the sum of the total outflow QF(I4,I5) to nodes N on iso-nodal line I5 have to be equal -QN4(I4)
* (it equal zero for collection node, certain values for source node)

Flow3(I4)..          sum(I3,QO(I3,I4)*z(I3,I4))-sum(I5,QF(I4,I5)*s(I4,I5)) =e= -QN4(I4) ;


* The difference between the sum of total collected inflow QF(I4,I5) at nodes N on iso-nodal line I4 minus
* the sum of the total outflow QG(I5,I6) to nodes N on iso-nodal line I6 have to be equal -QN5(I5)
* (it equal zero for collection node, certain values for source node)

Flow4(I5)..          sum(I4,QF(I4,I5)*s(I4,I5))-sum(I6,QG(I5,I6)*O(I5,I6)) =e= -QN5(I5);


* The difference between the sum of total collected inflow  QG(I5,I6) at nodes N on iso-nodal line I5 minus
* the sum of the total outflow QN(I6,I7) to nodes N on iso-nodal line I7 have to be equal  -QN6(I6)
* (it equal zero for collection node, certain values for source node)

Flow5(I6)..         sum(I5,QG(I5,I6)*O(I5,I6))-sum(I7,QN(I6,I7)*H(I6,I7)) =e= -QN6(I6);


* The sum of total collected inflow QN(I6,I7) at nodes N on iso-nodal line I6  to nodes N on iso-nodal line I7
*(Candidate WWTP Nodes) have to be equal to outflow (treated wastewater) at each WWTP node N on iso-nodal
*line I7.

Flow6(I7)..         sum(I6,QN(I6,I7)*H(I6,I7))=E=  QT(I7)*t(I7);

* The flow from each node N on iso-nodal line I1  must flow through one node on iso-nodal line I2

Conduct1(I1)..        sum(I2,x(I1,I2))=e= 1  ;


* The flow from each node N on iso-nodal line I2  must flow through one node on iso-nodal line I3
```

```
Conduct2(I2)..          sum(I3,y(I2,I3))=e= 1*(sum(I3,QC(I2,I3)))/(ABS((sum(I3,QC(I2,I3))-1))+1) ;

* The flow from each node N on iso-nodal line I3  must flow through one node on iso-nodal line I4

Conduct3(I3)..          sum(I4,z(I3,I4))=e= 1*(sum(I4,QO(I3,I4)))/(ABS((sum(I4,QO(I3,I4))-1))+1) ;

* The flow from each node N on iso-nodal line I4  must flow through one node on iso-nodal line I5

Conduct4(I4)..          sum(I5,s(I4,I5))=e= 1*(sum(I5,QF(I4,I5)))/(ABS((sum(I5,QF(I4,I5))-1))+1) ;

* The flow from each node N on iso-nodal line I5  must flow through one node on iso-nodal line I6

Conduct5(I5)..          sum(I6,O(I5,I6))=e= 1*(sum(I6,QG(I5,I6)))/(ABS((sum(I6,QG(I5,I6))-1))+1) ;

* The flow from each node N on iso-nodal line I6  must flow through one node on iso-nodal line I7
Conduct6(I6)..          sum(I7, H(I6,I7))=e= 1*(sum(I7,QN(I6,I7)))/(ABS((sum(I7,QN(I6,I7))-1))+1) ;

*--------------------------------------------------------------------------
* Upper bound for the flow through the sewer system have to be between maximum and
* minimum values for example : 4488 m3/d are upper value for the flow in sewer from node n on iso-nodal line i1
*  to node n on iso-nodal line i2
maxUpp(I1,I2)..       QS(I1,I2) =l= 4488.6*x(I1,I2);
maxUpp1(I2,I3)..      QC(I2,I3) =l= 13341.7*y(I2,I3);
maxUpp2(I3,I4)..      QO(I3,I4) =l= 19659.3*z(I3,I4);
maxUpp3(I4,I5)..      QF(I4,I5) =L= 25063.2*s(I4,I5);
maxUpp4(I5,I6)..      QG(I5,I6) =L= 64480*O(I5,I6);
maxUpp5(I6,I7)..      QN(I6,I7) =L= 72343.4*H(I6,I7);

*--------------------------------------------------------------------------

* Objective function which is a function of flow and length

cost1..     COST =e=
* costs associated with sewer system from node N on iso-nodal line I1 to node N on iso-nodal line I2
              sum((I1,I2), COSTLEN*L1(I1,I2)*QS(I1,I2)*x(I1,I2))

* costs associated with sewer system from node N on iso-nodal line I2 to node N on iso-nodal line I3
             +sum((I2,I3), COSTLEN*L2(I2,I3)*QC(I2,I3)*y(I2,I3))

* costs associated with sewer system from node N on iso-nodal line I3 to node N on iso-nodal line I4
             +sum((I3,I4), COSTLEN*L3(I3,I4)*QO(I3,I4)*z(I3,I4))

* costs associated with sewer system from node N on iso-nodal line I4 to node N on iso-nodal line I5
             +sum((I4,I5), COSTLEN*L4(I4,I5)*QF(I4,I5)*s(I4,I5))

* costs associated with sewer system from node N on iso-nodal line I5 to node N on iso-nodal line I6
             +sum((I5,I6), COSTLEN*L5(I5,I6)*QG(I5,I6)*O(I5,I6))

* costs associated with sewer system from node N on iso-nodal line I6 to node N on iso-nodal line I7
             +sum((I6,I7), COSTLEN*L6(I6,I7)*QN(I6,I7)*H(I6,I7))

* costs associated with treated wastewater at node N on iso-nodal line I7
             +SUM(I7, 0.0101*QT(I7)**2+895.9*QT(I7)+62838) ;

*------------------------------------------------------------------------
MODEL ZONE3M1 /ALL/;
*------------------------------------------------------------------------
SOLVE ZONE3M1 USING MINLP MINIMIZING COST;

*Here the model will use BARON solver to solve the problem
OPTION MINLP=BARON;
*------------------------------------------------------------------------
DISPLAY QS.L,QC.L,QO.L,QT.L,QF.L,QG.L,QN.L, cost.l, x.L, y.L,z.L,s.L,O.L,H.L,t.l ;
```

APPENDIX E

GAMS CODE USED TO SOLVE THE APPLICATION OF AN OPTIMIZATION MODEL FOR DESIGNING REGIONAL WASTEWATER SYSTEMS, DEVELOPED IN CHAPTER 7 FOR ZONE 3, M-2

```
*THIS IS MODEL FOR ZONE 3 -CASE STUDY- JIZAN REGION, KSA.
* M-2
*MODELED BY FAISAL ALFAISAL
*ADVISER. PROF.LARRY MAYS
*THURSDAY-07 MARCH


Sets
    I1    First iso-nodal line has one nodes                     /n31/
    I2    Second iso-nodal line have three nodes                 /n7/
    I3    Third iso-nodal line have three nodes                 /W-7/
    I4    Fourth iso-nodal line have one nodes                  /n30/
    I5    Fifth iso-nodal line have one nodes                   /W-30/
    I6    Sixth iso-nodal line have one nodes                    /n33/
    I7    Seventh iso-nodal line have one nodes                 /n24/
    I8    Eight iso-nodal line have one nodes                   /n14/
    I9    Nine iso-nodal line have three nodes                  /n11/
    I10   Ten iso-nodal line have three nodes                   /n5/
    I11   Eleven iso-nodal line have one nodes                 /W-5/
    I12   Twelve iso-nodal line have one nodes                 /n13/
    I13   Thirteen iso-nodal line have one nodes               /W-13/
    I14   Fourteen iso-nodal line have one nodes               /n10/
    I15   Fifteen iso-nodal line have one nodes               /n2/
    I16   Sixteen iso-nodal line have one nodes               /W-2/

    D   12 pipe diameters in mm    /PD300, PD375, PD450, PD525, PD600, PD675, PD750
                        PD900, PD1050,  PD1200,  PD1350,  PD1650    /
    P   1 pump          /PUMP1/


      ;

Parameters
    DIAIN(D) diameter (mm) of each pipe size
   /
     PD300      300
     PD375      375
     PD450      450
     PD525      525
     PD600      600
     PD675      675
     PD750      750
     PD900      900
     PD1050     1050
     PD1200     1200
     PD1350     1350
     PD1650     1650

/

  COSTPIPE(D)       cost ($) of pipe per m

   /
    PD300       99
    PD375       130
    PD450       165
    PD525       200
    PD600       235
    PD675       260
    PD750       310
    PD900       390
    PD1050      475
    PD1200      560
    PD1350      650
```

```
    PD1650      840

/

   COSTPUMP(P)      cost ($) of pumping head per meter
         /PUMP1    15000
         /

Table     QN1(I1,I2)  Wastewater produced  of nodes N on iso-nodal line 1

                    n7
          n31            0.104

* Zero value = collection node or certain value = source node
Table     QN2(I2,I3)  Transshipment at nodes N on iso-nodal line 2
                W-7
        n7        10941.1

* Zero value = collection node or certain value = source node
Table       QN3(I4,I5)   Transshipment at nodes N on iso-nodal line 3

                    W-30
            n30     2400.6

* Zero value = collection node or certain value = source node
Table     QN4(I6,I7)   Transshipment at nodes N on iso-nodal line 4
                 n24
            n33    0.084

* Zero value = collection node or certain value = source node

Table     QN5(I7,I9)   Transshipment at nodes N  on iso-nodal line 5

                n11
        n24       0.07

Table     QN6(I8,I10)   Transshipment at nodes N  on iso-nodal line 5

                n5
        n14       0.062

* Zero value = collection node or certain value = source node

Table     QN7(I9,I10)   Transshipment at nodes N  on iso-nodal line
               n5
*          n11      0.121
        n11      0.242

Table     QN8(I10,I11)   Transshipment at nodes N  on iso-nodal Line 5
                W-5
          n5       18377.1

Table     QN9(I14,I12)   Transshipment at nodes N  on iso-nodal Line 5
```

	n13
n10	0.098

Table QN10(I12,I13) Transshipment at nodes N on iso-nodal Line 5

	W-13
n13	7269.7

Table QN11(I15,I16) Transshipment at nodes N on iso-nodal Line 5

	W-2
n2	33354.9

Table L1(I1,I2) Wastewater Produced of nodes N on iso-nodal Line 1

	n7
n31	9000

* Zero value = collection node or certain value = source node

Table L2(I2,I3) Transshipment at nodes N on iso-nodal Line 2

	W-7
n7	0

* Zero value = collection node or certain value = source node

Table L3(I4,I5) Transshipment at nodes N on iso-nodal Line 3

	W-30
n30	0

* Zero value = collection node or certain value = source node

Table L4(I6,I7) Transshipment at nodes N on iso-nodal Line 4

	n24
n33	12700

* Zero value = collection node or certain value = source node

Table L5(I7,I9) Transshipment at nodes N on iso-nodal Line 5

	n11
n24	16000

Table L6(I8,I10) Transshipment at nodes N on iso-nodal Line 5

	n5
n14	17500

Table L7(I9,I10) Transshipment at nodes N on iso-nodal Line 5

	n5
n11	6000

Table L8(I10,I11) Transshipment at nodes N on iso-nodal Line 5

	W-5
n5	0

Table L9(I12,I13) Transshipment at nodes N on iso-nodal Line 5

```
               W-13
          n13       0


Table    L10(I14,I12)   Transshipment at nodes N  on iso-nodal Line 5
                n13
         n10      17000

Table    L11(I15,I16)   Transshipment at nodes N  on iso-nodal Line 5
               W-2
          n2       0




 ;


*THIS VALUE IS GREATER THAN TOTAL WASTEWATER PRODUCED AT SOURCE NODE I.
scalar    COSTLEN        Treatment Capacity     /300000/
          Md         Constant value        /3.21/
          N          Manning's value       /0.013/
          PI         Pai's value           /3.14/

;

*THESE ARE THE DECISION VARIABLES
Variables


             COST       The Optimum cost

;

Positive Variable
             L                length of certain pipes (at link (I1-I2). (I2-I3).and (I3-I4) for diameter D)
             Eleup            Elevations
             Elelo             Elevations
             XP

             ;


* Variable binary that will take value 1 if there is a pumpage linking node on iso-nodal line 1 to node on iso-nodal
line 2 and 0 otherwise
Binary Variable x(I1,I2)  ;
* Variable binary that will take value 1 if there is a pumpage linking node on iso-nodal line 6 to node on iso-nodal
line 7 and 0 otherwise
Binary Variable y(I6,I7)   ;
* Variable binary that will take value 1 if there is a pumpage linking node on iso-nodal line 7 to node on iso-nodal
line 9 and 0 otherwise
Binary Variable z(I7,I9)  ;
* Variable binary that will take value 1 if there is a pumpage linking node on iso-nodal line 8 to node on iso-nodal
line 9 and 0 otherwise
Binary Variable s(I8,I10)  ;
* Variable binary that will take value 1 if there is a pumpage linking node on iso-nodal line 9 to node on iso-nodal
line 10 and 0 otherwise
Binary Variable O(I9,I10)  ;
* Variable binary that will take value 1 if there is a pumpage linking node on iso-nodal line 14 to node on iso-
nodal line 15 and 0 otherwise
Binary Variable H(I14,I12)  ;



Equations
```

```
   cost1              Objective function

* Hydraulic constraints
   LENGTH1              length constraints from iso-nodal line I1 to iso-nodal line I2
   LENGTH2              length constraints from iso-nodal line I6 to iso-nodal line I7
   LENGTH3              length constraints from iso-nodal line I7 to iso-nodal line I9
   LENGTH4              length constraints from iso-nodal line I8 to iso-nodal line I9
   LENGTH5              length constraints from iso-nodal line I9 to iso-nodal line I10
   LENGTH6              length constraints from iso-nodal line I14 to iso-nodal line I15



   HYDRAULIC1           Hydraulic constraints from iso-nodal line I1  to iso-nodal line I3
   HYDRAULIC2           Hydraulic constraints from iso-nodal line I6  to iso-nodal line I7
   HYDRAULIC3           Hydraulic constraints from iso-nodal line I7  to iso-nodal line I9
   HYDRAULIC4           Hydraulic constraints from iso-nodal line I8  to iso-nodal line I9
   HYDRAULIC5           Hydraulic constraints from iso-nodal line I9  to iso-nodal line I10
   HYDRAULIC6           Hydraulic constraints from iso-nodal line I14 to iso-nodal line I15

      ElevJ1
      ElevJ2
      ElevJ3
      ElevJ4
      ElevJ5

;
*---------------------------------------------------------------------------
* The length required from each  iso-nodal  line (I) to the next iso-nodal line (I+1)

LENGTH1(I1,I2)..       sum(D, L(I1,I2,D))   =E=  L1(I1,I2);

LENGTH2(I6,I7)..       sum(D, L(I6,I7,D))   =E=  L4(I6,I7);

LENGTH3(I7,I9)..       sum(D, L(I7,I9,D))   =E=  L5(I7,I9);

LENGTH4(I8,I10)..       sum(D, L(I8,I10,D))   =E=  L6(I8,I10);

LENGTH5(I9,I10)..      sum(D, L(I9,I10,D))  =E=  L7(I9,I10);

LENGTH6(I14,I12)..     sum(D, L(I14,I12,D)) =E=  L10(I14,I12);



HYDRAULIC1(I1,I2)..       SUM(P,XP(I1,I2,P)*x(I1,I2))-
SUM(D,(((Md)**2*(n)**2*QN1(I1,I2)**2)*L(I1,I2,D))/((DIAIN(D)/1000)**(16/3)))   =G=  -(+(1200-
Eleup(I1,I2))-(800-Elelo(I1,I2)));

HYDRAULIC2(I6,I7)..       SUM(P,XP(I6,I7,P)*y(I6,I7))-
SUM(D,(((Md)**2*(n)**2*QN4(I6,I7)**2)*L(I6,I7,D))/((DIAIN(D)/1000)**(16/3)))    =G=  -(+(400-
Eleup(I6,I7))-(230-Elelo(I6,I7)));

HYDRAULIC3(I7,I9)..       SUM(P,XP(I7,I9,P)*z(I7,I9))-
SUM(D,(((Md)**2*(n)**2*QN5(I7,I9)**2)*L(I7,I9,D))/((DIAIN(D)/1000)**(16/3)))   =G=  -(+(230-
Eleup(I7,I9))-(110-Elelo(I7,I9)));

HYDRAULIC4(I8,I10)..       SUM(P,XP(I8,I10,P)*s(I8,I10))-
SUM(D,(((Md)**2*(n)**2*QN6(I8,I10)**2)*L(I8,I10,D))/((DIAIN(D)/1000)**(16/3)))   =G=  -(+(100-
Eleup(I8,I10))-(80-Elelo(I8,I10)));

HYDRAULIC5(I9,I10)..      SUM(P,XP(I9,I10,P)*O(I9,I10))-
SUM(D,(((Md)**2*(n)**2*QN7(I9,I10)**2)*L(I9,I10,D))/((DIAIN(D)/1000)**(16/3)))    =G=  -(+(110-
Eleup(I9,I10))-(80-Elelo(I9,I10)));
```

```
HYDRAULIC6(I14,I12)..     SUM(P,XP(I14,I12,P)*H(I14,I12))-
SUM(D,(((Md)**2*(n)**2*QN9(I14,I12)**2)*L(I14,I12,D))/((DIAIN(D)/1000)**(16/3)))   =G=  -(+(50-
Eleup(I14,I12))-(30-Elelo(I14,I12)));


ElevJ1(I6,I7)..            Elelo(I6,I7)=E=0;
ElevJ2(I7,I9)..            Elelo(I7,I9)=E=0;
ElevJ3(I8,I10)..           Elelo(I8,I10)=E=0;
ElevJ4(I9,I10)..           Elelo(I9,I10)=E=0;
ElevJ5(I14,I12)..          Elelo(I14,I12)=E=0;

* Objective function

cost1..       COST =e=
* costs associated with sewer system from node N on iso-nodal line I1 to node N on iso-nodal line I2
              + sum((I1,I2),(9000*Eleup(I1,I2)))
              + sum((I1,I2),SUM(P, COSTPUMP(P)*XP(I1,I2,P)*x(I1,I2)))

              + SUM((I1,I2),
SUM(D,COSTPIPE(D)*L(I1,I2,D)*100000000$(QN1(I1,I2)/((Pi/4)*((DIAIN(D)/1000)**(2))) < 0.6 OR
QN1(I1,I2)/((Pi/4)*((DIAIN(D)/1000)**(2))) > 2.6 )
                           +COSTPIPE(D)*L(I1,I2,D)$(QN1(I1,I2)/((Pi/4)*((DIAIN(D)/1000)**(2))) >= 0.6 OR
QN1(I1,I2)/((Pi/4)*((DIAIN(D)/1000)**(2))) <= 2.6 )))
* costs associated with sewer system from node N on iso-nodal line I6 to node N on iso-nodal line I7
              + sum((I6,I7),(9000*Eleup(I6,I7)))
              + sum((I6,I7),SUM(P, COSTPUMP(P)*XP(I6,I7,P)*y(I6,I7)))

              + SUM((I6,I7),
SUM(D,COSTPIPE(D)*L(I6,I7,D)*100000000$(QN4(I6,I7)/((Pi/4)*((DIAIN(D)/1000)**(2))) < 0.6 OR
QN4(I6,I7)/((Pi/4)*((DIAIN(D)/1000)**(2))) > 2.6 )
                           +COSTPIPE(D)*L(I6,I7,D)$(QN4(I6,I7)/((Pi/4)*((DIAIN(D)/1000)**(2))) >= 0.6 OR
QN4(I6,I7)/((Pi/4)*((DIAIN(D)/1000)**(2))) <= 2.6 )))


* costs associated with sewer system from node N on iso-nodal line I7 to node N on iso-nodal line I9
              + sum((I7,I9),(9000*Eleup(I7,I9)))
              + sum((I7,I9),SUM(P, COSTPUMP(P)*XP(I7,I9,P)*z(I7,I9)))

              + SUM((I7,I9),
SUM(D,COSTPIPE(D)*L(I7,I9,D)*100000000$(QN5(I7,I9)/((Pi/4)*((DIAIN(D)/1000)**(2))) < 0.6 OR
QN5(I7,I9)/((Pi/4)*((DIAIN(D)/1000)**(2))) > 2.6 )
                           +COSTPIPE(D)*L(I7,I9,D)$(QN5(I7,I9)/((Pi/4)*((DIAIN(D)/1000)**(2))) >= 0.6 OR
QN5(I7,I9)/((Pi/4)*((DIAIN(D)/1000)**(2))) <= 2.6 )))


* costs associated with sewer system from node N on iso-nodal line I8 to node N on iso-nodal line I10
              + sum((I8,I10),(9000*Eleup(I8,I10)))
              + sum((I8,I10),SUM(P, COSTPUMP(P)*XP(I8,I10,P)*s(I8,I10)))

              + SUM((I8,I10),
SUM(D,COSTPIPE(D)*L(I8,I10,D)*100000000$(QN6(I8,I10)/((Pi/4)*((DIAIN(D)/1000)**(2))) < 0.6 OR
QN6(I8,I10)/((Pi/4)*((DIAIN(D)/1000)**(2))) > 2.6 )
                           +COSTPIPE(D)*L(I8,I10,D)$(QN6(I8,I10)/((Pi/4)*((DIAIN(D)/1000)**(2))) >= 0.6
OR QN6(I8,I10)/((Pi/4)*((DIAIN(D)/1000)**(2))) <= 2.6 )))
* costs associated with sewer system from node N on iso-nodal line I9 to node N on iso-nodal line I10
              + sum((I9,I10),(9000*Eleup(I9,I10)))
              + sum((I9,I10),SUM(P, COSTPUMP(P)*XP(I9,I10,P)*O(I9,I10)))

              + SUM((I9,I10),
SUM(D,COSTPIPE(D)*L(I9,I10,D)*100000000$(QN7(I9,I10)/((Pi/4)*((DIAIN(D)/1000)**(2))) < 0.6 OR
QN7(I9,I10)/((Pi/4)*((DIAIN(D)/1000)**(2))) > 2.6 )
                           +COSTPIPE(D)*L(I9,I10,D)$(QN7(I9,I10)/((Pi/4)*((DIAIN(D)/1000)**(2))) >= 0.6
OR QN7(I9,I10)/((Pi/4)*((DIAIN(D)/1000)**(2))) <= 2.6 )))
```

```
* costs associated with sewer system from node N on iso-nodal line I14 to node N on iso-nodal line I12
                + sum((I14,I12),(9000*Eleup(I14,I12)))
                + sum((I14,I12),SUM(P, COSTPUMP(P)*XP(I14,I12,P)*H(I14,I12)))

                + SUM((I14,I12),
SUM(D,COSTPIPE(D)*L(I14,I12,D)*100000000$(QN9(I14,I12)/((Pi/4)*((DIAIN(D)/1000)**(2))) < 0.6 OR
QN9(I14,I12)/((Pi/4)*((DIAIN(D)/1000)**(2))) > 2.6 )
                              +COSTPIPE(D)*L(I14,I12,D)$(QN9(I14,I12)/((Pi/4)*((DIAIN(D)/1000)**(2))) >=
0.6 OR QN9(I14,I12)/((Pi/4)*((DIAIN(D)/1000)**(2))) <= 2.6 )))



* costs associated with treated wastewater at node N on iso-nodal line I3
                 +SUM((I2,I3), (0.0101*QN2(I2,I3)**2+895.9*QN2(I2,I3)+62838))

* costs associated with treated wastewater at node N on iso-nodal line I5
                 +SUM((I4,I5), (0.0101*QN3(I4,I5)**2+895.9*QN3(I4,I5)+62838))

* costs associated with treated wastewater at node N on iso-nodal line I11
                 +SUM((I10,I11), (0.0101*QN8(I10,I11)**2+895.9*QN8(I10,I11)+62838))

* costs associated with treated wastewater at node N on iso-nodal line I13
                 +SUM((I12,I13), (0.0101*QN10(I12,I13)**2+895.9*QN10(I12,I13)+62838))

* costs associated with treated wastewater at node N on iso-nodal line I16
                 +SUM((I15,I16), (0.0101*QN11(I15,I16)**2+895.9*QN11(I15,I16)+62838))
;
*---------------------------------------------------------------------------
MODEL ZONEM2 /ALL/;
*---------------------------------------------------------------------------
SOLVE ZONEM2 USING MINLP MINIMIZING COST;

*Here the model will use BARON solver to solve the problem
OPTION MINLP=BARON;
*---------------------------------------------------------------------------
DISPLAY L.L,Eleup.l,Elelo.l,COST.l, x.L,y.L, z.L,s.L,o.L, h.L, XP.L;
```

APPENDIX F

GAMS CODE USED TO SOLVE THE APPLICATION OF AN OPTIMIZATION MODEL FOR LAYOUT AND PIPE DESGIN STORM WATER SYSTEMS, DEVELOPED IN CHAPTER 8 FOR FIRST APPROACH, SCENARIO 2, M-3

```
*THIS IS MODEL FOR SIMULTANEOUSLY DETERMINING THE LAYOUT AND THE PIPE DESIGN FOR
STORM WATER SYSTEMS
* FIRST APPROACH.SCENARIO 2
*MIXED INTEGER NON-LINEAR PROGRAMMING (MINLP) IS USED TO MINIMIZE COSTS FOR
DETERMINISTIC MODEL.
*MODELED BY FAISAL ALFAISAL
*ADVISER. PROF.LARRY MAYS
*WED-02 MAY


Sets
    i   Manhole nodes on iso-nodal line I                  /m11, m12/
    j   Manhole nodes on iso-nodal line I+1                /m21, m22/
    k   Sewer outlet nodes on iso-nodal line I+2           /m31, m32/
    D   12 pipe diameters     /PD8, PD10, PD12, PD15, PD18, PD21, PD24, PD30/

     ;
Parameters

   Qin(i)           Inflow for manhole nodes on iso-nodal line I
     /   m11    5
         m12    5
       /

* Zero value = collection node or certain value = source node

    QI(j)           Inflow for manhole nodes on iso-nodal line I+1
     /   m21    5
         m22    5
               /

    DIAIN(D)      Commercial diameters in inches (in)

   /
    PD8        8
    PD10       10
    PD12       12
    PD15       15
    PD18       18
    PD21       21
    PD24       24
    PD30       30

/

  COSTPIPE(D)      Cost ($) of pipe per foot

   /

    PD8        34
    PD10       41
    PD12       47
    PD15       57
    PD18       70
    PD21       81
    PD24       102
    PD30       114

/
    ES1(i)  The surface elevations at manholes nodes on iso-nodal line I

  /      m11    100
         m12     98
/
```

```
   ES2(j)  The surface elevations at manholes nodes on iso-nodal line I+1

 /        m21    90.25
          m22    91.75

/

   ES3(k)  The surface elevations at manhole nodes (outlet nodes) on iso-nodal line I+2

 /        m31    85.50
          m32    84.75

/

 ;

* These values are used in manning's equation constraints which defined as scalar values in GAMS program
scalar
            Md       Constant value        /2.16/
             N       Manning's value       /0.013/
             PI      Pai's value       /3.14/

  ;

Table RHS(i,j)  The length required  from manhole nodes on  iso-nodal  line I to manhole nodes on  iso-nodal  line
I+1
            m21          m22
  m11        900          900
  m12        900          900
  ;

Table RHSS(j,k)  The length required from manhole nodes on iso-nodal line I+1 to manhole nodes on outlet nodes
I+2
             m31          m32
   m21        900          900
   m22        900          900

 ;

*These are the decision variables
Variables

   COST       The Optimum cost  ;

Positive Variable
QS(i,j)      flow rate in sewer system from manhole nodes on iso-nodal line I to manhole nodes on iso-nodal line
I+1
QC(j,k)      flow rate in sewer system from manhole nodes on iso-nodal line I+1 to outlet nodes I+2
Qout(k)     flow rate  leaves outlet nodes I+2
L              length of certain pipes (at link (I, I+1) and (I+1, I+2) for diameter D)
Eleup       Crown elevation upstream
Elelo         Crown elevation downstream
Slope(i,j)    Slope from manhole nodes on iso-nodal line I to manhole nodes on iso-nodal line I+1
Slope1(j,k)    Slope from manhole nodes on iso-nodal line I+1 to outlet nodes I+2

               ;

* Variable binary that will take value 1 if there is a sewer linking node I to node I+1 and 0 otherwise
Binary Variable x(i,j)  ;

* Variable binary that will take value 1 if there is a sewer linking node I+1 to node I+2 and 0 otherwise
Binary Variable y(j,k)   ;
```

Equations
    cost1 Objective function

*--
* Subject to
* Layout Constraints
* 1) Continuity constraint:
    Con1 Continuity constraint at node I
    Con2 Continuity constraint at node I+1
    Con3 Continuity constraint at node I+2

*--
* 2) Connectivity models
* enforcing each node should release one path.

Conduct1 Conductivity constraint from manhole nodes on iso-nodal line I to manhole nodes on iso-nodal line I+1
Conduct2 Conductivity constraint from manhole nodes on iso-nodal line I+1 to outlet nodes I+2

*--
* Pipe Design Constraints
*Commercial diameters constraints
* 1) Length Constants

LENGTH length constraints from manhole nodes on iso-nodal line i to manhole nodes on iso-nodal line I+1
LENGTH1 length constraints from manhole nodes on iso-nodal line I+1 to outlet nodes I+2

* 2) Manning's Equation Constants
HYDRAULIC Manning's Equation from manhole nodes on iso-nodal line I to manhole nodes on iso-nodal line I+1
HYDRAULIC1 Manning's Equation from manhole nodes on iso-nodal line I+1 to outlet nodes I+2

*--
* 3) Upper and lower bound for slopes in the system
MaxElv1 Maximum slope is written for each possible pipe connection from manholes nodes on iso-nodal line i to manholes on iso-nodal line I+1

MinElv1 Minimum slope is written for each possible pipe connection from manholes nodes on iso-nodal line i to manholes on iso-nodal line I+1

MaxElv2 Maximum slope is written for each possible pipe connection from manholes nodes on iso-nodal line I+1 to outlet nodes I+2

MinElv2 Minimum slope is written for each possible pipe connection from manholes nodes on iso-nodal line I+1 to outlet nodes I+2

Slopeq1 Slope equation for each possible pipe connection from manholes nodes on iso-nodal line i to manholes on iso-nodal line I+1

Slopeq2 Slope equation for each possible pipe connection from manholes nodes on iso-nodal line I+1 to outlet nodes I+2

*--
* 4) Continues slope constraints to ensure that slope continues in the downstream direction.
Upper
Upper1

* 5) Minimum pipe cover depth constraints, it assumed to be 3 ft minimum cover depth
ElevUP
ElevUP1

* 6) Junction constraints
ElevJ

```
*7) Tie-ins constraints
ElTi

* Upper and lower bound
    maxUpp         Upper Bounded Eq for maximum amount of flow allowed  on arc I and I+1
    maxUpp1        Upper Bounded Eq for maximum amount of flow allowed  on arc I+1 and I+2
    minLo           Lower Bounded Eq for minimum amount of flow allowed  on arc i and I+1
    minLo1          Lower Bounded Eq for minimum amount of flow allowed  on arc I+1 and I+2
;

* The produce flow QR(i) at manhole nodes on iso-nodal line i should be equal to the sum of the conveyed flow
QS(i,j)  from manhole nodes on iso-nodal line i to manhole on iso-nodal line I+1

Con1(i)..                Qin(i) =E=sum(j, QS(i,j)*x(i,j)) ;


* The difference between the sum of the total collected inflow QS(i,j) at manholes nodes on iso-nodal line I+1
minus the sum of the total outflow QC(j,k) to outlet nodes on iso-nodal line I+2 have to be equal zero

Con2(j)..            sum(i, QS(i,j)*x(i,j))-sum(k, QC(j,k)*y(j,k)) =e= -QI(j) ;

* The sum of the conveyed flow QC(j,k) from manholes nodes on iso-nodal line I+1 to outlet nodes I+2, have to
be equal to outflow Qout(k) at each candidate outlet manhole nodes on iso-nodal line I+2.

Con3(k)..                sum(j, QC(j,k)*y(j,k)) =e= Qout(k);


* The flow from each manhole nodes on iso-nodal line I must flow through one manhole nodes on iso-nodal line
I+1
Conduct1(i)..            sum(j, x(i,j))=e=1 ;

* The flow from each manhole nodes on iso-nodal line I+1 must flow through one outlet nodes I+2 QC(j,k).
Conduct2(j)..            sum(k, y(j,k))=e= 1*(sum(k, QC(j,k)))/(ABS((sum(k, QC(j,k))-1))+1) ;


* The length required from manhole nodes on iso-nodal line i to manhole nodes on iso-nodal line I+1
LENGTH(i,j)..          sum(D, L(i,j,D)) =E=  RHS(i,j)*x(i,j);

* the length required from manhole nodes on iso-nodal line I+1 to outlet nodes I+2
LENGTH1(j,k)..         sum(D, L(j,k,D)) =E=  RHSS(j,k)*y(j,k);

*------------------------------------------------------------------------------
* Manning’s Equation is written for each possible pipe connection from manholes nodes on iso-nodal line i to
manholes on iso-nodal line I+1


HYDRAULIC(i,j)..
SUM(D,(((Md)**2*(n)**2*QS(i,j)**2)*L(i,j,D)*x(i,j))/((DIAIN(D)/12)**(16/3)))=E=((ES1(i)-Eleup(i,j))-
(ES2(j)-Elelo(i,j)))*x(i,j);

* Minimum and maximum slope is written for each possible pipe connection from manholes nodes on iso-nodal
line i to manholes on iso-nodal line I+1

MaxElv1(i,j)..                Slope(i,j)=L=0.2+1000000*(1-x(i,j));
MinElv1(i,j)..                Slope(i,j)+1000000*(1-x(i,j))=G=0.005 ;


*Slope equation is written for pipe connection from manholes nodes on iso-nodal line i to manholes on iso-nodal
line I+1
Slopeq1(i,j)..              ((ES1(i)-Eleup(i,j))-(ES2(j)-Elelo(i,j)))*x(i,j)=E= Slope(i,j)*RHS(i,j)*x(i,j);
```

* Manning's Equation is written for each possible pipe connection from manholes nodes on iso-nodal line i to manholes on iso-nodal line I+1

```
HYDRAULIC1(j,k)..
SUM(D,(((Md)**2*(n)**2*QC(j,k)**2)*L(j,k,D)*y(j,k))/((DIAIN(D)/12)**(16/3)))=E=((ES2(j)-Eleup(j,k))-(ES3(k)-Elelo(j,k)))*y(j,k);
```

*Minimum and maximum slope is written for each possible pipe connection from manholes nodes on iso-nodal line I+1 to manholes on outlet nodes I+2

```
MaxElv2(j,k)..                    Slope1(j,k)=L=0.2+10000000*(1-y(j,k));
MinElv2(j,k)..                    Slope1(j,k)+10000000*(1-y(j,k))=G=0.005;
```

*Slope equation is written for pipe connection from manholes nodes on iso-nodal line I+1 to manholes on outlet nodes I+2

```
Slopeq2(j,k)..                    ((ES2(j)-Eleup(j,k))-(ES3(k)-Elelo(j,k)))*y(j,k)=E=Slope1(j,k)*RHSS(j,k)*y(j,k);
```

*--

* Continues slope constraints to ensure that slope continues in the downstream direction.

```
Upper(i,j)..              Elelo(i,j)*x(i,j) =G= Eleup(i,j)*x(i,j);
Upper1(j,k)..              Elelo(j,k)*y(j,k) =G= Eleup(j,k)*y(j,k);
```

*--

* Minimum pipe cover depth constraints, it assumed to be 3 ft minimum cover depth

```
ElevUP(i,j)..          Eleup(i,j) =G=3*x(i,j);
ElevUP1(j,k)..          Eleup(j,k) =G=3*y(j,k);
```

*--

* Junction constraints where the cutting depth upstream of manholes m_{n+1} on the iso-nodal line n+1 have to be greater or equal to the cutting depth downstream of manholes m_n on the iso-nodal line n.

```
ElevJ(i,j,k)..          Eleup(j,k)=G= Elelo(i,j)*y(j,k);
```

*--

* Tie-ins to existing sewer systems must also be defined if the sewer being designed connects to outlet sewer; So, the new crown elevation must be lower than the crown elevation of the existing pipe ET.

```
ElTi(j,k)..               ES3(k)-Elelo(j,k)=G=80;
```

*--

* Upper and lower bound for the flow through the sewer system have to be between maximum and minimum values

```
maxUpp(i,j)..          QS(i,j) =l=  5*x(i,j);
maxUpp1(j,k)..        Qc(j,k) =l= 20*y(j,k);
minLo(i,j)..            QS(i,j) =g= 0*x(i,j);
minLo1(j,k)..          Qc(j,k) =g= 0*y(j,k);
```

*--

*The objective function for scenario 2 is to determine simultaneously layout and pipe design stormwater system

*Objective function

```
COST1..       COST =E=
```

* Costs of manhole per unit depth at manhole nodes on iso-nodal line i.

```
SUM((i,j), (1818.2*Eleup(i,j)-1000)*x(i,j))
```

* Costs of pipeline per unit foot from manhole nodes on iso-nodal line i to manhole nodes on iso-nodal line I+1

```
                 +  SUM((i,j), SUM(D,COSTPIPE(D)*L(i,j,D)*x(i,j)))

* Costs of manhole per unit depth at manhole nodes on iso-nodal line I+1.
                  + SUM((j,k), (1818.2*Eleup(j,k)-1000)*y(j,k))

* Costs of pipeline per unit foot from manhole nodes on iso-nodal line I+1 to outlet nodes I+2

                 + sum ((j,k), SUM(D,COSTPIPE(D)*L(j,k,D)*y(j,k)))


*------------------------------------------------------------------------
MODEL examfaisal /ALL/;
*------------------------------------------------------------------------
SOLVE examfaisal USING MINLP MINIMIZING COST;

*------------------------------------------------------------------------
DISPLAY QS.L,QC.L,Qout.L, cost.l, x.L, y.L,L.L,Eleup.L,Elelo.L,Slope.l,Slope1.l  ;
```

APPENDIX G

GAMS CODE USED TO SOLVE THE APPLICATION OF AN OPTIMIZATION MODEL FOR LAYOUT AND PIPE DESGIN STORM WATER SYSTEMS, DEVELOPED IN CHAPTER 8 FOR SECOND APPROACH, S-2, M-3

```
*THIS IS MODEL FOR SIMULTANEOUSLY DETERMINING THE LAYOUT AND THE PIPE DESIGN FOR
STORM WATER SYSTEMS
* SECOND APPROACH.SCENARIO-2
*MIXED INTEGER NON-LINEARPROGRAMMING(MINLP) IS USED TO MINIMIZE COSTS FOR
DETERMINISTIC MODEL.
*MODELED BY FAISAL ALFAISAL
*ADVISER. PROF.LARRY MAYS
*WED-02 MAY


Sets
    i   Manhole nodes on iso-nodal line I                /m11, m12/
    j   Manhole nodes on iso-nodal line I+1              /m21, m22/
    k   Sewer outlet nodes on iso-nodal line I+2         /m31, m32/
    D   12 pipe diameters     /PD8, PD10, PD12, PD15, PD18, PD21, PD24, PD30/
     ;
Parameters

   Qin(i)          Inflow for manhole nodes on iso-nodal line I
     /  m11    5
        m12    5
       /

* Zero value = collection node or certain value = source node

    QI(j)          Inflow for manhole nodes on iso-nodal line I+1
     /   m21    5
         m22    5

             /
    DIA(D)      Commercial diameters in inches (in)

   /

    PD8        8
    PD10       10
    PD12       12
    PD15       15
    PD18       18
    PD21       21
    PD24       24
    PD30       30

/

  COSTPIPE(D)       Cost ($) of pipe per foot

   /

    PD8        34
    PD10       41
    PD12       47
    PD15       57
    PD18       70
    PD21       81
    PD24       102
    PD30       114

/
    ES1(i)  The surface elevations at manholes nodes on iso-nodal line I

  /       m11    100
          m12    98
/
```

```
   ES2(j)  The surface elevations at manholes nodes on iso-nodal line I+1

 /       m21    90.25
         m22    91.75

/

   ES3(k)  The surface elevations at manhole nodes (outlet nodes) on iso-nodal line I+2

 /       m31    85.50
         m32    84.75

/

 ;

* These values are used in manning's equation constraints which defined as scalar values in GAMS program
scalar
         Md        Constant value       /2.16/
         N           Manning's value       /0.013/
         VMAX      Maximum velocity in pipe  /15/
         VMIN       Minimum velocity in pipe  /3/


  ;

Table RHS(i,j)  The length required  from manhole nodes on  iso-nodal  line I to manhole nodes on  iso-nodal  line
I+1
           m21         m22
  m11       900         900
  m12       900         900


  ;

Table RHSS(j,k)  The length required from manhole nodes on iso-nodal line I+1 to manhole nodes on outlet nodes
I+2
           m31         m32
   m21       900         900
   m22       900         900

 ;
*These are the decision variables
Variables

COST       The Optimum cost  ;

Positive Variable
QS(i,j)      flow rate in sewer system from manhole nodes on iso-nodal line I to manhole nodes on iso-nodal line
I+1
QC(j,k)     flow rate in sewer system from manhole nodes on iso-nodal line I+1 to outlet nodes I+2
Qout(k)   flow rate  leaves outlet nodes I+2
EleI(i)     Crown elevations at upstream nodes on iso-nodal line I
EleJ(j)     Crown elevations at upstream nodes on iso-nodal line I+1
EleK(K)    Crown elevations at upstream nodes on iso-nodal line I+2
DiffE(i,j)         The different elevation from manhole nodes on iso-nodal line I to manhole nodes on iso-nodal line
I+1
DiffE1(j,k)       The different elevation from manhole nodes on iso-nodal line I+1 to manhole nodes on iso-nodal
line I+2
Slope(i,j)       Slope from manhole nodes on iso-nodal line I to manhole nodes on iso-nodal line I+1
Slope1(j,k)      Slope from manhole nodes on iso-nodal line I+1 to outlet nodes I+2
```

```
               ;

* Variable binary that will a sign one commercial diameter for that link is selected
Binary Variable CanD(i,j,d);
Binary Variable CanD1(j,k,d);

* Variable binary that will take value 1 if there is a sewer linking node I to node I+1 and 0 otherwise
Binary Variable x(i,j)  ;
* Variable binary that will take value 1 if there is a sewer linking node I+1 to node I+2 and 0 otherwise
Binary Variable y(j,k)   ;


Equations
    cost1            Objective function

*-----------------------------------------------------------------------------------------------------------------------
* Subject to
* Layout Constraints
*    1)  Continuity constraint:

    con1             Continuity constraint at iso-nodal line I
    con2             Continuity constraint at iso-nodal line I+1
    con3             Continuity constraint at iso-nodal line I+2

*-----------------------------------------------------------------------------------------------------------------------
*  2) Connectivity models
* Enforcing each node should release one path.

    Conduct1        Conductivity constraint from manholes on iso-nodal line I  to manholes on iso-nodal line I+1
    Conduct2        Conductivity constraint from manholes on iso-nodal line I+1  to manholes on sewer outlet
nodes

    DiamCons         Constraint for selection of a single diameter
    DiamCons1       Constraint for selection of a single diameter

*-----------------------------------------------------------------------------------------------------------------------
* Pipe Design Constraints
* Commercial diameters constraints
*  1)  Manning's Equation Constants

HYDRAULIC     Manning's Equation from manhole nodes on iso-nodal line I to manhole nodes on iso-nodal line
I+1
HYDRAULIC1   Manning's Equation from manhole nodes on iso-nodal line I+1 to outlet nodes I+2
EleDiff           Different crown elevations of manholes on iso-nodal line I  to manholes on iso-nodal line I+1
EleDiff1         Different crown elevations of manholes on iso-nodal line I+1  to manholes on sewer outlet nodes
SlopeEq          Slope equation applied for Pipe from manholes on iso-nodal line I  to manholes on iso-nodal line
I+1
Slope1Eq        Slope equation applied for Pipe from manholes on iso-nodal line I+1  to manholes on sewer outlet
nodes I+2

*-------------------------------------------------------------------------------------------------------------------------
* 2) Upper and lower bound for slopes in the system

MaxVelocity      Maximum velocity applied for Pipe from manholes on iso-nodal line I  to manholes on iso-nodal
line I+1
MinVelocity      Minimum velocity applied for Pipe from manholes on iso-nodal line I  to manholes on iso-nodal
line I+1
MaxVelocity1     Maximum velocity applied for Pipe from manholes on iso-nodal line I+1  to manholes on sewer
outlet nodes I+2
 MinVelocity1     Minimum velocity applied for Pipe from manholes on iso-nodal line I+1  to manholes on sewer
outlet nodes
```

*---
* 5) Minimum pipe cover depth constraints, it assumed to be 3 ft minimum cover depth

CoverDepth Minimum Cover depth for Pipe from manholes on iso-nodal line I to manholes on iso-nodal line I+1
CoverDepth1 Minimum Cover depth for Pipe from manholes on iso-nodal line I+1 to manholes on sewer outlet nodes I+2

*---
* 4) Continues slope constraints to ensure that slope continues in the downstream direction.

ConSlop Continues slope for pipe from manholes on iso-nodal line I to manholes on iso-nodal line I+1
ConSlop1 Continues slope for pipe from manholes on iso-nodal line I+1 to manholes on sewer outlet nodes i+2

*---

*7) Tie-ins constraints
ElTi

*---
* Upper and lower bound
maxUpp Upper Bounded Eq for maximum amount of flow allowed on arc I and I+1
maxUpp1 Upper Bounded Eq for maximum amount of flow allowed on arc I+1 and I+2
minLo Lower Bounded Eq for minimum amount of flow allowed on arc I and I+1
minLo1 Lower Bounded Eq for minimum amount of flow allowed on arc I+1 and I+2
;

* The produce flow QR(i) at manhole nodes on iso-nodal line i should be equal to the sum of the conveyed flow QS(i,j) from manhole nodes on iso-nodal line i to manhole on iso-nodal line I+1

Con1(i).. Qin(i) =E=sum(j, QS(i,j)*x(i,j)) ;

* The difference between the sum of the total collected inflow QS(i,j) at manholes nodes on iso-nodal line I+1 minus the sum of the total outflow QC(j,k) to outlet nodes on iso-nodal line I+2 have to be equal zero

Con2(j).. sum(i, QS(i,j)*x(i,j))-sum(k, QC(j,k)*y(j,k)) =e= -QI(j) ;

* The sum of the conveyed flow QC(j,k) from manholes nodes on iso-nodal line I+1 to outlet nodes I+2, have to be equal to outflow Qout(k) at each candidate outlet manhole nodes on iso-nodal line I+2.

Con3(k).. sum(j, QC(j,k)*y(j,k)) =e= Qout(k);

DiamCons(i,j).. sum(D, CanD(i,j,D)) =E= x(i,j);
DiamCons1(j,k).. sum(D, CanD1(j,k,D)) =E= y(j,k);

* The flow from each manhole nodes on iso-nodal line I must flow through one manhole nodes on iso-nodal line I+1
Conduct1(i).. sum(j, x(i,j))=e=1 ;

* The flow from each manhole nodes on iso-nodal line I+1 must flow through one outlet nodes I+2 QC(j,k).
Conduct2(j).. sum(k, y(j,k))=e= 1*(sum(k, QC(j,k)))/(ABS((sum(k, QC(j,k))-1))+1) ;

*---
* Pipe Design Constraints

```
HYDRAULIC(i,j)..
SUM(D,(((Md)**2*(n)**2*QS(i,j)**2)*RHS(i,j)*x(i,j))/((DIA(D)/12)**(16/3))*CanD(i,j,D))=L=(DiffE(i,j))*x(i,j
);
EleDiff(i,j)..                DiffE(i,j) =E= ((ES1(i) - EleI(i)) - (ES2(j) - EleJ(j)))*x(i,j);
SlopeEq(i,j)..                   Slope(i,j) =E=  (DiffE(i,j)/RHS(i,j))*x(i,j);


HYDRAULIC1(j,k)..
SUM(D,(((Md)**2*(n)**2*QC(j,k)**2)*RHSS(j,k)*y(j,k))/((DIA(D)/12)**(16/3))*CanD1(j,k,D))=L=(DiffE1(j,k
))*y(j,k);
EleDiff1(j,k)..               DiffE1(j,k) =E= ((ES2(j) - EleJ(j)) - (ES3(k) - EleK(K)))*y(j,k);
Slope1Eq(j,k)..                  Slope1(j,k) =E=  (DiffE1(j,k)/RHSS(j,k))*y(j,k) ;


*Minimum and maximum velocity is written for each possible pipe connection from manholes nodes on iso-nodal
line i to manholes on iso-nodal line I+1

MaxVelocity(i,j)..               QS(i,j)=G= 2.5*sum(D, (3.14/4*(DIA(D)/12)**2)*CanD(i,j,D)) ;
MinVelocity(i,j)..               QS(i,j)=L= 12* sum(D, (3.14/4*(DIA(D)/12)**2)*CanD(i,j,D)) ;

MaxVelocity1(j,k)..               QC(j,k)=G= 2.5*sum(D, (3.14/4*(DIA(D)/12)**2)*CanD1(j,k,D)) ;
MinVelocity1(j,k)..               QC(j,k)=L= 12* sum(D, (3.14/4*(DIA(D)/12)**2)*CanD1(j,k,D)) ;

*-------------------------------------------------------------------------------
* Continues slopes
ConSlop(i,j)..            EleJ(j)*x(i,j)=G= EleI(i)*x(i,j);
ConSlop1(j,k)..             EleK(K)*y(j,k)=G= EleJ(j)*y(j,k);
*---------------------------------------------------------------------------
* Minimum cover depth
CoverDepth(i,j)..          EleI(i) =G=3*x(i,j);
CoverDepth1(j,k)..          EleJ(j) =G=3*y(j,k);

*---------------------------------------------------------------------------
* Tie-ins constraints
ElTi(j,k)..              ES3(k)-EleK(K)=G=80;


*---------------------------------------------------------------------------

maxUpp(i,j)..        QS(i,j) =l= 5*x(i,j);
maxUpp1(j,k)..       QC(j,k) =l= 20*y(j,k);
minLo(i,j)..        QS(i,j) =g= 0*x(i,j);
minLo1(j,k)..        QC(j,k) =g= 0*y(j,k);


COST1..       COST =E=
* Costs of pipeline per unit foot from manhole nodes on iso-nodal line i to manhole nodes on iso-nodal line I+1

                  SUM((i,j,d), (CanD(i,j,d))*COSTPIPE(d)*RHS(i,j))

* Costs of pipeline per unit foot from manhole nodes on iso-nodal line I+1 to outlet nodes I+2

                 + SUM((j,k,d), (CanD1(j,k,d))*COSTPIPE(d)*RHSS(j,k))

* Costs of manhole per unit depth at manhole nodes on iso-nodal line I.

                 + SUM( (i,j),  (1818.2*EleI(i)-1000)*x(i,j))

* Costs of manhole per unit depth at manhole nodes on iso-nodal line I+1.

                 + SUM( (j,k),  (1818.2*EleJ(j)-1000)*y(j,k))

;
```

```
MODEL examfaisal /ALL/;
*--------------------------------------------------------------------------
SOLVE examfaisal USING MINLP MINIMIZING COST;

*HERE THE MODEL WILL USE BARON SOLVER TO SOLVE THE PROBLEM
OPTION MINLP=BARON;

DISPLAY QS.L,QC.L,Qout.L, cost.l, x.L,
y.L,CanD.l,CanD1.l,EleI.L,EleJ.L,EleK.L,DiffE.L,DiffE1.L,Slope.l,Slope1.l  ;
```

APPENDIX H

GAMS CODE USED TO SOLVE THE APPLICATION OF AN OPTIMIZATION MODEL FOR PIPE DESIGN STORM WATER SYSTEMS, DEVELOPED IN CHAPTER 9, M-4

```
*THIS IS MODEL FOR PIPE DESGIN STORMWATER SYSTEM
* NONLINEAR PROGRAMMING (NLP) IS USED TO MINIMIZE COSTS FOR DETERMINISTIC MODEL.
*MODELED BY FAISAL ALFAISAL
*ADVISER. PROF.LARRY MAYS
*FRD-11 MAY


Sets
    B2   Manhole B2                    /MH-B2/
    B1   Manhole B1                    /MH-B1/
    L7   Manhole L7                    /MH-L7/
    L6   Manhole L6                    /MH-L6/
    L5   Manhole L5                    /MH-L5/
    L4   Manhole L4                    /MH-L4/
    L3   Manhole L3                    /MH-L3/
    A2   Manhole A2                    /MH-A2/
    A1   Manhole A1                    /MH-A1/
    L2   Manhole L2                    /MH-L2/
    L1   Manhole L1                    /MH-L1/
    S2   Manhole S2                    /MH-S2/
    S9   Manhole S9                    /MH-S9/
    S8   Manhole S8                    /MH-S8/
    S7   Manhole S7                    /MH-S7/
    S6   Manhole S6                    /MH-S6/
    S5   Manhole S5                    /MH-S5/
    S4   Manhole S4                    /MH-S4/
    S3   Manhole S3                    /MH-S3/
    S1   Manhole S1                    /MH-S1/
    S0   Manhole S0                    /MH-S0/

   D   12 pipe diameters     /  PD24,PD30 , PD36, PD42, PD48   /
;

Parameters


    DIAIN(D) diameter (inches) of each pipe size
   /

    PD24       24
    PD30       30
    PD36       36
    PD42       42
    PD48       48
/

  COSTPIPE(D)       cost ($) of pipe per foot

   /

    PD24       97
    PD30       110
    PD36       137
    PD42       164
    PD48       181
/

 ;

*These values are used in manning's equation constraints which defined as scalar values in GAMS program
scalar
         Md         Constant value        /2.16/
         N         Manning's value       /0.013/
         PI         Pai's value       /3.14/
```

;
* The length required from manhole nodes on iso-nodal line I to manhole nodes on iso-nodal line I+1
Table RHS(B2,B1) LENGTH
            MH-B1
  MH-B2   404

Table RHSS(B1,L6) LENGTH
              MH-L6
   MH-B1   267

Table RHS2(L7,L6) LENGTH
              MH-L6
   MH-L7   553

Table RHS3(L6,L5) LENGTH
              MH-L5
   MH-L6   403

Table RHS4(L5,L4) LENGTH
              MH-L4
   MH-L5   464

Table RHS5(L4,L3) LENGTH
              MH-L3
   MH-L4   441

Table RHS6(A2,A1) LENGTH
              MH-A1
   MH-A2   220

Table RHS7(A1,L3) LENGTH
              MH-L3
   MH-A1   332

Table RHS8(L3,L2) LENGTH
              MH-L2
   MH-L3   392

Table RHS9(L2,L1) LENGTH
              MH-L1
   MH-L2   389

Table RHS10(L1,S2) LENGTH
             MH-S2
   MH-L1   367

Table RHS11(S9,S8) LENGTH
              MH-S8
   MH-S9   257
Table RHS12(S8,S7) LENGTH
              MH-S7
   MH-S8   72

Table RHS13(S7,S6) LENGTH
              MH-S6
   MH-S7   434

Table RHS14(S6,S5) LENGTH
              MH-S5

MH-S6	358

Table RHS15(S5,S4) LENGTH

	MH-S4
MH-S5	77

Table RHS16(S4,S3) LENGTH

	MH-S3
MH-S4	114

Table RHS17(S3,S2) LENGTH

	MH-S2
MH-S3	340

Table RHS18(S2,S1) LENGTH

	MH-S1
MH-S2	623

Table RHS19(S1,S0) LENGTH

	MH-S0
MH-S1	70

*Design flow from manhole nodes on iso-nodal line I to manhole nodes on iso-nodal line I+1

Table Q1(B2,B1) PIPE FLOW

	MH-B1
MH-B2	11.2

Table Q2(B1,L6) PIPE FLOW

	MH-L6
MH-B1	11.2

Table Q3(L7,L6) PIPE FLOW

	MH-L6
MH-L7	15.1

Table Q4(L6,L5) PIPE FLOW

	MH-L5
MH-L6	32.6

Table Q5(L5,L4) PIPE FLOW

	MH-L4
MH-L5	32.6

Table Q6(L4,L3) PIPE FLOW

	MH-L3
MH-L4	32.6

Table Q7(A2,A1) PIPE FLOW

	MH-A1
MH-A2	12.4

Table Q8(A1,L3) PIPE FLOW

	MH-L3
MH-A1	12.4

Table Q9(L3,L2) PIPE FLOW

	MH-L2
MH-L3	52.5

```
Table Q10(L2,L1) PIPE FLOW
              MH-L1
   MH-L2        52.5

Table Q11(L1,S2) PIPE FLOW
               MH-S2
   MH-L1         52.5

Table Q12(S9,S8) PIPE FLOW
              MH-S8
   MH-S9        13.4

Table Q13(S8,S7) PIPE FLOW
              MH-S7
   MH-S8        13.4

Table Q14(S7,S6) PIPE FLOW
              MH-S6
   MH-S7        28.5

Table Q15(S6,S5) PIPE FLOW
              MH-S5
   MH-S6        28.5

Table Q16(S5,S4) PIPE FLOW
              MH-S4
   MH-S5        38.1

Table Q17(S4,S3) PIPE FLOW
              MH-S3
   MH-S4        38.1

Table Q18(S3,S2) PIPE FLOW
              MH-S2
   MH-S3        38.1

Table Q19(S2,S1) PIPE FLOW
              MH-S1
   MH-S2        108.5

Table Q20(S1,S0) PIPE FLOW
              MH-S0
   MH-S1        108.5
 ;

*THESE ARE THE DECISION VARIABLES
Variables


    COST       The Optimum cost  ;

Positive Variable
              L              Length of certain pipes for diameter D
              Eleup        Upstream crown elevations
              Elelo         Downstream crown elevations

              ;

Equations
    cost1              Objective function

*----------------------------------------------------------------------------------------------------------------------------------------
----
* Pipe Design Constraints
```

*Commercial diameters constraints
* 1) Length Constants

LENGTH
LENGTH1
LENGTH2
LENGTH3
LENGTH4
LENGTH5
LENGTH6
LENGTH7
LENGTH8
LENGTH9
LENGTH10
LENGTH11
LENGTH12
LENGTH13
LENGTH14
LENGTH15
LENGTH16
LENGTH17
LENGTH18
LENGTH19

*--
----
* 2) Manning's Equation Constants
HYDRAULIC
HYDRAULIC1
HYDRAULIC2
HYDRAULIC3
HYDRAULIC4
HYDRAULIC5
HYDRAULIC6
HYDRAULIC7
HYDRAULIC8
HYDRAULIC9
HYDRAULIC10
HYDRAULIC11
HYDRAULIC12
HYDRAULIC13
HYDRAULIC14
HYDRAULIC15
HYDRAULIC16
HYDRAULIC17
HYDRAULIC18
HYDRAULIC19
*--
----
* 3) Minimum cover depth for each upstream and downstream crown elevations.

ElevUP
ElevLO
ElevUP1
ElevLO1
ElevUP2
ElevLO2
ElevUP3
ElevLO3
ElevUP4
ElevLO4
ElevUP5
ElevLO5
ElevUP6

ElevLO6
ElevUP7
ElevLO7
ElevUP8
ElevLO8
ElevUP9
ElevLO9
ElevUP10
ElevLO10
ElevUP11
ElevLO11
ElevUP12
ElevLO12
ElevUP13
ElevLO13
ElevUP14
ElevLO14
ElevUP15
ElevLO15
ElevUP16
ElevLO16
ElevUP17
ElevLO17
ElevUP18
ElevLO18
ElevUP19

*---
----
* 4) Junction constraints

ElevJ
ElevJ1
ElevJ2
ElevJ3
ElevJ4
ElevJ5
ElevJ6
ElevJ7
ElevJ8
ElevJ9
ElevJ10
ElevJ11
ElevJ12
ElevJ13
ElevJ14
ElevJ15
ElevJ16
ElevJ17
ElevJ18

*---
----
*5) Tie-ins constraints

ElevJ5J5

*---
----

*6) A signing one commercial diameter for each pipe constraints
DIM
DIM1
DIM2

```
        DIM3
        DIM4
        DIM5
        DIM6
        DIM7
        DIM8
        DIM9
        DIM10
        DIM11
        DIM12
        DIM13
        DIM14
        DIM15
        DIM16
        DIM17
        DIM18
        DIM19

 ;

* The length required
LENGTH(B2,B1)..          sum(D, L(B2,B1,D))=E=  RHS(B2,B1);

LENGTH1(B1,L6)..          sum(D, L(B1,L6,D))=E= RHSS(B1,L6);

LENGTH2(L7,L6)..          sum(D, L(L7,L6,D))=E= RHS2(L7,L6);

LENGTH3(L6,L5)..          sum(D, L(L6,L5,D))=E= RHS3(L6,L5);

LENGTH4(L5,L4)..          sum(D, L(L5,L4,D))=E= RHS4(L5,L4);

LENGTH5(L4,L3)..          sum(D, L(L4,L3,D))=E= RHS5(L4,L3);

LENGTH6(A2,A1)..          sum(D, L(A2,A1,D))=E= RHS6(A2,A1);

LENGTH7(A1,L3)..          sum(D, L(A1,L3,D))=E= RHS7(A1,L3);

LENGTH8(L3,L2)..          sum(D, L(L3,L2,D))=E= RHS8(L3,L2);

LENGTH9(L2,L1)..          sum(D, L(L2,L1,D))=E= RHS9(L2,L1);

LENGTH10(L1,S2)..          sum(D, L(L1,S2,D))=E= RHS10(L1,S2);

LENGTH11(S9,S8)..          sum(D, L(S9,S8,D))=E= RHS11(S9,S8);

LENGTH12(S8,S7)..          sum(D, L(S8,S7,D))=E= RHS12(S8,S7);

LENGTH13(S7,S6)..          sum(D, L(S7,S6,D))=E= RHS13(S7,S6);

LENGTH14(S6,S5)..          sum(D, L(S6,S5,D))=E= RHS14(S6,S5);

LENGTH15(S5,S4)..          sum(D, L(S5,S4,D))=E= RHS15(S5,S4);

LENGTH16(S4,S3)..          sum(D, L(S4,S3,D))=E= RHS16(S4,S3);

LENGTH17(S3,S2)..          sum(D, L(S3,S2,D))=E= RHS17(S3,S2);

LENGTH18(S2,S1)..          sum(D, L(S2,S1,D))=E= RHS18(S2,S1);
```

```
LENGTH19(S1,S0)..          sum(D, L(S1,S0,D))=E= RHS19(S1,S0);
*---------------------------------------------------------------------------

DIM(B2,B1)..          SUM ( D, L(B2,B1,D)/(L(B2,B1,D)+0.0001)) =L= 1;
DIM1(B1,L6)..         SUM ( D, L(B1,L6,D)/(L(B1,L6,D)+0.0001)) =L= 1;
DIM2(L7,L6)..         SUM ( D, L(L7,L6,D)/(L(L7,L6,D)+0.0001)) =L= 1 ;
DIM3(L6,L5)..         SUM ( D, L(L6,L5,D)/(L(L6,L5,D)+0.0001)) =L= 1 ;
DIM4(L5,L4)..         SUM ( D, L(L5,L4,D)/(L(L5,L4,D)+0.0001)) =L= 1 ;
DIM5(L4,L3)..         SUM ( D, L(L4,L3,D)/(L(L4,L3,D)+0.0001)) =L= 1 ;
DIM6(L3,L2)..         SUM ( D, L(L3,L2,D)/(L(L3,L2,D)+0.0001)) =L= 1 ;
DIM7(A2,A1)..         SUM ( D, L(A2,A1,D)/(L(A2,A1,D)+0.0001)) =L= 1 ;
DIM8(A1,L3)..         SUM ( D, L(A1,L3,D)/(L(A1,L3,D)+0.0001)) =L= 1 ;
DIM9(L2,L1)..         SUM ( D, L(L2,L1,D)/(L(L2,L1,D)+0.0001)) =L= 1 ;
DIM10(L1,S2)..        SUM ( D, L(L1,S2,D)/(L(L1,S2,D)+0.0001)) =L= 1 ;
DIM11(S9,S8)..        SUM ( D, L(S9,S8,D)/(L(S9,S8,D)+0.0001)) =L= 1 ;
DIM12(S8,S7)..        SUM ( D, L(S8,S7,D)/(L(S8,S7,D)+0.0001)) =L= 1 ;
DIM13(S7,S6)..        SUM ( D, L(S7,S6,D)/(L(S7,S6,D)+0.0001)) =L= 1 ;
DIM14(S6,S5)..        SUM ( D, L(S6,S5,D)/(L(S6,S5,D)+0.0001)) =L= 1 ;
DIM15(S5,S4)..        SUM ( D, L(S5,S4,D)/(L(S5,S4,D)+0.0001)) =L= 1 ;
DIM16(S4,S3)..        SUM ( D, L(S4,S3,D)/(L(S4,S3,D)+0.0001)) =L= 1 ;
DIM17(S3,S2)..        SUM ( D, L(S3,S2,D)/(L(S3,S2,D)+0.0001)) =L= 1 ;
DIM18(S2,S1)..        SUM ( D, L(S2,S1,D)/(L(S2,S1,D)+0.0001)) =L= 1 ;
DIM19(S1,S0)..        SUM ( D, L(S1,S0,D)/(L(S1,S0,D)+0.0001)) =L= 1 ;



*---------------------------------------------------------------------------


HYDRAULIC(B2,B1)..
SUM(D,(((Md)**2*(n)**2*Q1(B2,B1)**2)*L(B2,B1,D))/((DIAIN(D)/12)**(16/3)))=E=  (+(1257.76-
Eleup(B2,B1))-(1257.34-Elelo(B2,B1)));

HYDRAULIC1(B1,L6)..
SUM(D,(((Md)**2*(n)**2*Q2(B1,L6)**2)*L(B1,L6,D))/((DIAIN(D)/12)**(16/3)))=E=  (+(1257.34-
Eleup(B1,L6))-(1256.81-Elelo(B1,L6)));

HYDRAULIC2(L7,L6)..
SUM(D,(((Md)**2*(n)**2*Q3(L7,L6)**2)*L(L7,L6,D))/((DIAIN(D)/12)**(16/3)))=E=  (+(1257.93-
Eleup(L7,L6))-(1256.81-Elelo(L7,L6)));

HYDRAULIC3(L6,L5)..
SUM(D,(((Md)**2*(n)**2*Q4(L6,L5)**2)*L(L6,L5,D))/((DIAIN(D)/12)**(16/3)))=E=  (+(1256.81-
Eleup(L6,L5))-(1254.36-Elelo(L6,L5)));

HYDRAULIC4(L5,L4)..
SUM(D,(((Md)**2*(n)**2*Q5(L5,L4)**2)*L(L5,L4,D))/((DIAIN(D)/12)**(16/3)))=E=  (+(1254.36-
Eleup(L5,L4))-(1252.59-Elelo(L5,L4)));

HYDRAULIC5(L4,L3)..
SUM(D,(((Md)**2*(n)**2*Q6(L4,L3)**2)*L(L4,L3,D))/((DIAIN(D)/12)**(16/3)))=E=  (+(1252.59-
Eleup(L4,L3))-(1251.13-Elelo(L4,L3)));

HYDRAULIC6(A2,A1)..
SUM(D,(((Md)**2*(n)**2*Q7(A2,A1)**2)*L(A2,A1,D))/((DIAIN(D)/12)**(16/3)))=E=  (+(1254.01-
Eleup(A2,A1))-(1252.79-Elelo(A2,A1)));

HYDRAULIC7(A1,L3)..
SUM(D,(((Md)**2*(n)**2*Q8(A1,L3)**2)*L(A1,L3,D))/((DIAIN(D)/12)**(16/3)))=E=  (+(1252.79-
Eleup(A1,L3))-(1251.13-Elelo(A1,L3)));
```

```
HYDRAULIC8(L3,L2)..
SUM(D,(((Md)**2*(n)**2*Q9(L3,L2)**2)*L(L3,L2,D))/((DIAIN(D)/12)**(16/3)))=E=  (+(1251.13-
Eleup(L3,L2))-(1248.64-Elelo(L3,L2)));

HYDRAULIC9(L2,L1)..
SUM(D,(((Md)**2*(n)**2*Q10(L2,L1)**2)*L(L2,L1,D))/((DIAIN(D)/12)**(16/3)))=E= (+(1248.64-
Eleup(L2,L1))-(1247.54-Elelo(L2,L1)));

HYDRAULIC10(L1,S2)..
SUM(D,(((Md)**2*(n)**2*Q11(L1,S2)**2)*L(L1,S2,D))/((DIAIN(D)/12)**(16/3)))=E= (+(1247.54-
Eleup(L1,S2))-(1246.43-Elelo(L1,S2)));

HYDRAULIC11(S9,S8)..
SUM(D,(((Md)**2*(n)**2*Q12(S9,S8)**2)*L(S9,S8,D))/((DIAIN(D)/12)**(16/3)))=E= (+(1251.59-
Eleup(S9,S8))-(1251.12-Elelo(S9,S8)));

HYDRAULIC12(S8,S7)..
SUM(D,(((Md)**2*(n)**2*Q13(S8,S7)**2)*L(S8,S7,D))/((DIAIN(D)/12)**(16/3)))=E= (+(1251.12-
Eleup(S8,S7))-(1250.93-Elelo(S8,S7)));

HYDRAULIC13(S7,S6)..
SUM(D,(((Md)**2*(n)**2*Q14(S7,S6)**2)*L(S7,S6,D))/((DIAIN(D)/12)**(16/3)))=E= (+(1250.93-
Eleup(S7,S6))-(1249.20-Elelo(S7,S6)));

HYDRAULIC14(S6,S5)..
SUM(D,(((Md)**2*(n)**2*Q15(S6,S5)**2)*L(S6,S5,D))/((DIAIN(D)/12)**(16/3)))=E= (+(1249.20-
Eleup(S6,S5))-(1247.86-Elelo(S6,S5)));

HYDRAULIC15(S5,S4)..
SUM(D,(((Md)**2*(n)**2*Q16(S5,S4)**2)*L(S5,S4,D))/((DIAIN(D)/12)**(16/3)))=E= (+(1247.86-
Eleup(S5,S4))-(1247.64-Elelo(S5,S4)));

HYDRAULIC16(S4,S3)..
SUM(D,(((Md)**2*(n)**2*Q17(S4,S3)**2)*L(S4,S3,D))/((DIAIN(D)/12)**(16/3)))=E= (+(1247.64-
Eleup(S4,S3))-(1246.80-Elelo(S4,S3)));

HYDRAULIC17(S3,S2)..
SUM(D,(((Md)**2*(n)**2*Q18(S3,S2)**2)*L(S3,S2,D))/((DIAIN(D)/12)**(16/3)))=E= (+(1246.80-
Eleup(S3,S2))-(1246.43-Elelo(S3,S2)));

HYDRAULIC18(S2,S1)..
SUM(D,(((Md)**2*(n)**2*Q19(S2,S1)**2)*L(S2,S1,D))/((DIAIN(D)/12)**(16/3)))=E= (+(1246.43-
Eleup(S2,S1))-(1244.74-Elelo(S2,S1)));

HYDRAULIC19(S1,S0)..
SUM(D,(((Md)**2*(n)**2*Q20(S1,S0)**2)*L(S1,S0,D))/((DIAIN(D)/12)**(16/3)))=E= (+(1244.74-
Eleup(S1,S0))-(1243.11-Elelo(S1,S0)));


ElevUP(B2,B1)..          Eleup(B2,B1)=G=3;
ElevLO(B2,B1)..          Elelo(B2,B1)=G=3 ;

ElevUP1(B1,L6)..          Eleup(B1,L6)=G=3;
ElevLO1(B1,L6)..         Elelo(B1,L6)=G=3 ;

ElevUP2(L7,L6)..          Eleup(L7,L6)=G=3;
ElevLO2(L7,L6)..         Elelo(L7,L6)=G=3 ;

ElevUP3(L6,L5)..          Eleup(L6,L5)=G=3;
ElevLO3(L6,L5)..         Elelo(L6,L5)=G=3 ;

ElevUP4(L5,L4)..          Eleup(L5,L4)=G=3;
```

```
ElevLO4(L5,L4)..          Elelo(L5,L4)=G=3 ;

ElevUP5(L4,L3)..          Eleup(L4,L3)=G=3;
ElevLO5(L4,L3)..          Elelo(L4,L3)=G=3 ;

ElevUP6(A2,A1)..          Eleup(A2,A1)=G=3;
ElevLO6(A2,A1)..          Elelo(A2,A1)=G=3 ;

ElevUP7(A1,L3)..          Eleup(A1,L3)=G=3;
ElevLO7(A1,L3)..          Elelo(A1,L3)=G=3 ;

ElevUP8(L3,L2)..          Eleup(L3,L2)=G=3;
ElevLO8(L3,L2)..          Elelo(L3,L2)=G=3 ;

ElevUP9(L2,L1)..          Eleup(L2,L1)=G=3;
ElevLO9(L2,L1)..          Elelo(L2,L1)=G=3 ;

ElevUP10(L1,S2)..         Eleup(L1,S2)=G=3;
ElevLO10(L1,S2)..         Elelo(L1,S2)=G=3 ;

ElevUP11(S9,S8)..         Eleup(S9,S8)=G=3;
ElevLO11(S9,S8)..         Elelo(S9,S8)=G=3 ;

ElevUP12(S8,S7)..         Eleup(S8,S7)=G=3;
ElevLO12(S8,S7)..         Elelo(S8,S7)=G=3 ;

ElevUP13(S7,S6)..         Eleup(S7,S6)=G=3;
ElevLO13(S7,S6)..         Elelo(S7,S6)=G=3 ;

ElevUP14(S6,S5)..         Eleup(S6,S5)=G=3;
ElevLO14(S6,S5)..         Elelo(S6,S5)=G=3 ;

ElevUP15(S5,S4)..         Eleup(S5,S4)=G=3;
ElevLO15(S5,S4)..         Elelo(S5,S4)=G=3 ;

ElevUP16(S4,S3)..         Eleup(S4,S3)=G=3;
ElevLO16(S4,S3)..         Elelo(S4,S3)=G=3 ;

ElevUP17(S3,S2)..         Eleup(S3,S2)=G=3;
ElevLO17(S3,S2)..         Elelo(S3,S2)=G=3 ;

ElevUP18(S2,S1)..         Eleup(S2,S1)=G=3;
ElevLO18(S2,S1)..         Elelo(S2,S1)=G=3 ;

ElevUP19(S1,S0)..         Eleup(S1,S0)=G=3;



ElevJ(B2,B1,L6)..         Eleup(B1,L6) =G= Elelo(B2,B1);
ElevJ1(B1,L6,L5)..        Eleup(L6,L5) =G= Elelo(B1,L6);
ElevJ2(L7,L6,L5)..        Eleup(L6,L5) =G= Elelo(L7,L6);
ElevJ3(L6,L5,L4)..        Eleup(L5,L4) =G= Elelo(L6,L5);
ElevJ4(L5,L4,L3)..        Eleup(L4,L3) =G= Elelo(L5,L4);
ElevJ6(L4,L3,L2)..        Eleup(L3,L2) =G= Elelo(L4,L3);
ElevJ5(A2,A1,L3)..        Eleup(A1,L3) =G= Elelo(A2,A1);

ElevJ7(A1,L3,L2)..        Eleup(L3,L2) =G= Elelo(A1,L3);
ElevJ8(L3,L2,L1)..        Eleup(L2,L1) =G= Elelo(L3,L2);

ElevJ9(L1,S2,L2)..        Eleup(L1,S2) =G= Elelo(L2,L1);

ElevJ16(L1,S2,S1)..       Eleup(S2,S1) =G= Elelo(L1,S2);
ElevJ10(S9,S8,S7)..       Eleup(S8,S7) =G= Elelo(S9,S8);
ElevJ11(S8,S7,S6)..       Eleup(S7,S6) =G= Elelo(S8,S7);
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```
ElevJ12(S7,S6,S5)..        Eleup(S6,S5) =G= Elelo(S7,S6);
ElevJ13(S6,S5,S4)..        Eleup(S5,S4) =G= Elelo(S6,S5);
ElevJ14(S5,S4,S3)..        Eleup(S4,S3) =G= Elelo(S5,S4);

ElevJ15(S3,S2,S4)..        Eleup(S3,S2) =G= Elelo(S4,S3);

ElevJ17(S3,S2,S1)..        Eleup(S2,S1) =G= Elelo(S3,S2);
ElevJ18(S2,S1,S0)..        Eleup(S1,S0) =G= Elelo(S2,S1);


ElevJ5J5(S1,S0)..         (1243.11-Elelo(S1,S0))=G=1239;


*Objective function
COST1..       COST =e=


* Total Cost of pipeline per foot


                     + SUM((B2,B1), (1818.2*Eleup(B2,B1)-1000))


                     + SUM((B2,B1),
SUM(D,COSTPIPE(D)*L(B2,B1,D)*100000000$(Q1(B2,B1)/((Pi/4)*((DIAIN(D)/12)**(2))) < 3 OR
Q1(B2,B1)/((Pi/4)*((DIAIN(D)/12)**(2))) > 15 )
                                  +COSTPIPE(D)*L(B2,B1,D)$(Q1(B2,B1)/((Pi/4)*((DIAIN(D)/12)**(2))) >= 3 OR
Q1(B2,B1)/((Pi/4)*((DIAIN(D)/12)**(2))) <= 15 )))


* Total Cost of pipeline per foot


                    + sum ((B1,L6), (1818.2*Eleup(B1,L6)-1000))
                    + sum ((B1,L6),
SUM(D,COSTPIPE(D)*L(B1,L6,D)*10000000$(Q2(B1,L6)/((Pi/4)*((DIAIN(D)/12)**(2))) < 3 OR
Q2(B1,L6)/((Pi/4)*((DIAIN(D)/12)**(2))) > 15 )
                                  +COSTPIPE(D)*L(B1,L6,D)$(Q2(B1,L6)/((Pi/4)*((DIAIN(D)/12)**(2))) >= 3 OR
Q2(B1,L6)/((Pi/4)*((DIAIN(D)/12)**(2))) <= 15 )))


                    + sum ((L7,L6), (1818.2*Eleup(L7,L6)-1000))

                    + sum ((L7,L6),
SUM(D,COSTPIPE(D)*L(L7,L6,D)*10000000$(Q3(L7,L6)/((Pi/4)*((DIAIN(D)/12)**(2))) < 3 OR
Q3(L7,L6)/((Pi/4)*((DIAIN(D)/12)**(2))) > 15 )
                                  +COSTPIPE(D)*L(L7,L6,D)$(Q3(L7,L6)/((Pi/4)*((DIAIN(D)/12)**(2))) >= 3 OR
Q3(L7,L6)/((Pi/4)*((DIAIN(D)/12)**(2))) <= 15 )))

                    + sum ((L6,L5), (1818.2*Eleup(L6,L5)-1000))




                    + sum ((L6,L5),
SUM(D,COSTPIPE(D)*L(L6,L5,D)*10000000$(Q4(L6,L5)/((Pi/4)*((DIAIN(D)/12)**(2))) < 3 OR
Q4(L6,L5)/((Pi/4)*((DIAIN(D)/12)**(2))) > 15 )
                                  +COSTPIPE(D)*L(L6,L5,D)$(Q4(L6,L5)/((Pi/4)*((DIAIN(D)/12)**(2))) >= 3 OR
Q4(L6,L5)/((Pi/4)*((DIAIN(D)/12)**(2))) <= 15 )))

                    + sum ((L5,L4), (1818.2*Eleup(L5,L4)-1000))
```

```
                  + sum ((L5,L4),
SUM(D,COSTPIPE(D)*L(L5,L4,D)*10000000$(Q5(L5,L4)/((Pi/4)*((DIAIN(D)/12)**(2))) < 3 OR
Q5(L5,L4)/((Pi/4)*((DIAIN(D)/12)**(2))) > 15 )
                              +COSTPIPE(D)*L(L5,L4,D)$(Q5(L5,L4)/((Pi/4)*((DIAIN(D)/12)**(2))) >= 3 OR
Q5(L5,L4)/((Pi/4)*((DIAIN(D)/12)**(2))) <= 15 )))

                  + sum ((L4,L3), (1818.2*Eleup(L4,L3)-1000))

                  + sum ((L4,L3),
SUM(D,COSTPIPE(D)*L(L4,L3,D)*10000000$(Q6(L4,L3)/((Pi/4)*((DIAIN(D)/12)**(2))) < 3  OR
Q6(L4,L3)/((Pi/4)*((DIAIN(D)/12)**(2))) > 15 )
                              +COSTPIPE(D)*L(L4,L3,D)$(Q6(L4,L3)/((Pi/4)*((DIAIN(D)/12)**(2))) >= 3  OR
Q6(L4,L3)/((Pi/4)*((DIAIN(D)/12)**(2))) <= 15 )))

                  + sum ((A2,A1), (1818.2*Eleup(A2,A1)-1000))


                  + sum ((A2,A1),
SUM(D,COSTPIPE(D)*L(A2,A1,D)*10000000$(Q7(A2,A1)/((Pi/4)*((DIAIN(D)/12)**(2))) < 3 OR
Q7(A2,A1)/((Pi/4)*((DIAIN(D)/12)**(2))) > 15 )
                              +COSTPIPE(D)*L(A2,A1,D)$(Q7(A2,A1)/((Pi/4)*((DIAIN(D)/12)**(2))) >= 3 OR
Q7(A2,A1)/((Pi/4)*((DIAIN(D)/12)**(2))) <= 15 )))

                  + sum ((A1,L3), (1818.2*Eleup(A1,L3)-1000))



                  + sum ((A1,L3),
SUM(D,COSTPIPE(D)*L(A1,L3,D)*10000000$(Q8(A1,L3)/((Pi/4)*((DIAIN(D)/12)**(2))) < 3 OR
Q8(A1,L3)/((Pi/4)*((DIAIN(D)/12)**(2))) > 15 )
                              +COSTPIPE(D)*L(A1,L3,D)$(Q8(A1,L3)/((Pi/4)*((DIAIN(D)/12)**(2))) >= 3 OR
Q8(A1,L3)/((Pi/4)*((DIAIN(D)/12)**(2))) <= 15 )))


                  + sum ((L3,L2), (1818.2*Eleup(L3,L2)-1000))


                  + sum ((L3,L2),
SUM(D,COSTPIPE(D)*L(L3,L2,D)*10000000$(Q9(L3,L2)/((Pi/4)*((DIAIN(D)/12)**(2))) < 3  OR
Q9(L3,L2)/((Pi/4)*((DIAIN(D)/12)**(2))) > 15 )
                              +COSTPIPE(D)*L(L3,L2,D)$(Q9(L3,L2)/((Pi/4)*((DIAIN(D)/12)**(2))) >= 3  OR
Q9(L3,L2)/((Pi/4)*((DIAIN(D)/12)**(2))) <= 15 )))

                  + sum ((L2,L1), (1818.2*Eleup(L2,L1)-1000))

                  + sum ((L2,L1),
SUM(D,COSTPIPE(D)*L(L2,L1,D)*10000000$(Q10(L2,L1)/((Pi/4)*((DIAIN(D)/12)**(2))) < 3 OR
Q10(L2,L1)/((Pi/4)*((DIAIN(D)/12)**(2))) > 15 )
                              +COSTPIPE(D)*L(L2,L1,D)$(Q10(L2,L1)/((Pi/4)*((DIAIN(D)/12)**(2))) >= 3 OR
Q10(L2,L1)/((Pi/4)*((DIAIN(D)/12)**(2))) <= 15 )))

                  + sum ((L1,S2), (1818.2*Eleup(L1,S2)-1000))

                  + sum ((L1,S2),
SUM(D,COSTPIPE(D)*L(L1,S2,D)*10000000$(Q11(L1,S2)/((Pi/4)*((DIAIN(D)/12)**(2))) < 3 OR
Q11(L1,S2)/((Pi/4)*((DIAIN(D)/12)**(2))) > 15 )
                              +COSTPIPE(D)*L(L1,S2,D)$(Q11(L1,S2)/((Pi/4)*((DIAIN(D)/12)**(2))) >= 3 OR
Q11(L1,S2)/((Pi/4)*((DIAIN(D)/12)**(2))) <= 15 )))
                 + sum ((S9,S8), (1818.2*Eleup(S9,S8)-1000))
                  + sum ((S9,S8),
SUM(D,COSTPIPE(D)*L(S9,S8,D)*10000000$(Q12(S9,S8)/((Pi/4)*((DIAIN(D)/12)**(2))) < 3 OR
Q12(S9,S8)/((Pi/4)*((DIAIN(D)/12)**(2))) > 15 )
                              +COSTPIPE(D)*L(S9,S8,D)$(Q12(S9,S8)/((Pi/4)*((DIAIN(D)/12)**(2))) >= 3 OR
Q12(S9,S8)/((Pi/4)*((DIAIN(D)/12)**(2))) <= 15 )))
```

```
                  + sum ((S8,S7), (1818.2*Eleup(S8,S7)-1000))


                  + sum ((S8,S7),
SUM(D,COSTPIPE(D)*L(S8,S7,D)*10000000$(Q13(S8,S7)/((Pi/4)*((DIAIN(D)/12)**(2))) < 3 OR
Q13(S8,S7)/((Pi/4)*((DIAIN(D)/12)**(2))) > 15 )
                                   +COSTPIPE(D)*L(S8,S7,D)$(Q13(S8,S7)/((Pi/4)*((DIAIN(D)/12)**(2))) >= 3 OR
Q13(S8,S7)/((Pi/4)*((DIAIN(D)/12)**(2))) <= 15 )))
                  + sum ((S7,S6), (1818.2*Eleup(S7,S6)-1000))

                  + sum ((S7,S6),
SUM(D,COSTPIPE(D)*L(S7,S6,D)*10000000$(Q14(S7,S6)/((Pi/4)*((DIAIN(D)/12)**(2))) < 5 OR
Q14(S7,S6)/((Pi/4)*((DIAIN(D)/12)**(2))) > 15 )
                                   +COSTPIPE(D)*L(S7,S6,D)$(Q14(S7,S6)/((Pi/4)*((DIAIN(D)/12)**(2))) >= 5 OR
Q14(S7,S6)/((Pi/4)*((DIAIN(D)/12)**(2))) <= 15 )))
                  + sum ((S6,S5), (1818.2*Eleup(S6,S5)-1000))
                  + sum ((S6,S5),
SUM(D,COSTPIPE(D)*L(S6,S5,D)*10000000$(Q15(S6,S5)/((Pi/4)*((DIAIN(D)/12)**(2))) < 3 OR
Q15(S6,S5)/((Pi/4)*((DIAIN(D)/12)**(2))) > 15 )
                                   +COSTPIPE(D)*L(S6,S5,D)$(Q15(S6,S5)/((Pi/4)*((DIAIN(D)/12)**(2))) >= 3 OR
Q15(S6,S5)/((Pi/4)*((DIAIN(D)/12)**(2))) <= 15 )))
                  + sum ((S5,S4), (1818.2*Eleup(S5,S4)-1000))

                  + sum ((S5,S4),
SUM(D,COSTPIPE(D)*L(S5,S4,D)*10000000$(Q16(S5,S4)/((Pi/4)*((DIAIN(D)/12)**(2))) < 3.5 OR
Q16(S5,S4)/((Pi/4)*((DIAIN(D)/12)**(2))) > 15 )
                                   +COSTPIPE(D)*L(S5,S4,D)$(Q16(S5,S4)/((Pi/4)*((DIAIN(D)/12)**(2))) >= 3.5
OR Q16(S5,S4)/((Pi/4)*((DIAIN(D)/12)**(2))) <= 15 )))
                  + sum ((S4,S3), (1818.2*Eleup(S4,S3)-1000))

                  + sum ((S4,S3),
SUM(D,COSTPIPE(D)*L(S4,S3,D)*10000000$(Q17(S4,S3)/((Pi/4)*((DIAIN(D)/12)**(2))) < 3.5 OR
Q17(S4,S3)/((Pi/4)*((DIAIN(D)/12)**(2))) > 15 )
                                   +COSTPIPE(D)*L(S4,S3,D)$(Q17(S4,S3)/((Pi/4)*((DIAIN(D)/12)**(2))) >= 3.5
OR Q17(S4,S3)/((Pi/4)*((DIAIN(D)/12)**(2))) <= 15 )))
                  + sum ((S3,S2), (1818.2*Eleup(S3,S2)-1000))
                  + sum ((S3,S2),
SUM(D,COSTPIPE(D)*L(S3,S2,D)*10000000$(Q18(S3,S2)/((Pi/4)*((DIAIN(D)/12)**(2))) < 3 OR
Q18(S3,S2)/((Pi/4)*((DIAIN(D)/12)**(2))) > 15 )
                                  +COSTPIPE(D)*L(S3,S2,D)$(Q18(S3,S2)/((Pi/4)*((DIAIN(D)/12)**(2))) >= 3 OR
Q18(S3,S2)/((Pi/4)*((DIAIN(D)/12)**(2))) <= 15 )))
                  + sum ((S2,S1), (1818.2*Eleup(S2,S1)-1000))
                  + sum ((S2,S1),
SUM(D,COSTPIPE(D)*L(S2,S1,D)*10000000$(Q19(S2,S1)/((Pi/4)*((DIAIN(D)/12)**(2))) < 3 OR
Q19(S2,S1)/((Pi/4)*((DIAIN(D)/12)**(2))) > 15 )
                                   +COSTPIPE(D)*L(S2,S1,D)$(Q19(S2,S1)/((Pi/4)*((DIAIN(D)/12)**(2))) >= 3 OR
Q19(S2,S1)/((Pi/4)*((DIAIN(D)/12)**(2))) <= 15 )))
                 + sum ((S1,S0), (1818.2*Eleup(S1,S0)-1000))
                  + sum ((S1,S0),
SUM(D,COSTPIPE(D)*L(S1,S0,D)*100000000$(Q20(S1,S0)/((Pi/4)*((DIAIN(D)/12)**(2))) < 3 OR
Q20(S1,S0)/((Pi/4)*((DIAIN(D)/12)**(2))) > 15 )
                                   +COSTPIPE(D)*L(S1,S0,D)$(Q20(S1,S0)/((Pi/4)*((DIAIN(D)/12)**(2))) >= 3 OR
Q20(S1,S0)/((Pi/4)*((DIAIN(D)/12)**(2))) <= 15 )))

;

*----------------------------------------------------------------------
MODEL examfaisal /ALL/;
*----------------------------------------------------------------------
SOLVE examfaisal USING NLP MINIMIZING COST;

*----------------------------------------------------------------------
```

```
DISPLAY L.L,Eleup.l,Elelo.l,COST.l ;
```

Results

1. Through GDX

```
execute_unload 'results.gdx',
L.L,Eleup.l,Elelo.l,COST.l ;
```

1.2 To XLS using gdxxrw

```
execute 'gdxxrw.exe results.gdx var=L.L rng=Length!';
execute 'gdxxrw.exe results.gdx var=Eleup.l rng=US_crown_Elev!';
execute 'gdxxrw.exe results.gdx var=Elelo.l rng=DS_crown_Elev!';
execute 'gdxxrw.exe results.gdx var=COST.l rng=cost!';
```