Water Supply Infrastructure Modeling and Control under

Extreme Drought and/or Limited Power Availability

by

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ABSTRACT

The phrase water-energy nexus is commonly used to describe the inherent and critical interdependencies between the electric power system and the water supply systems (WSS). The key interdependencies between the two systems are the power plant's requirement of water for the cooling cycle and the water system's need of electricity for pumping for water supply. While previous work has considered the dependency of WSS on the electrical power, this work incorporates into an optimization-simulation framework, consideration of the impact of short and long-term limited availability of water and/or electrical energy.

This research focuses on the WSS facet of the multi-faceted optimization and control mechanism developed for an integrated water – energy nexus system under U.S. National Science Foundation (NSF) project 029013-0010 CRISP Type 2 – Resilient cyber-enabled electric energy and water infrastructures modeling and control under extreme mega drought scenarios. A WSS conveys water from sources (such as lakes, rivers, dams etc.) to the treatment plants and then to users via the water distribution systems (WDS) and/or water supply canal systems (WSCS). Optimization-simulation methodologies are developed for the real-time operation of WSS under critical conditions of limited electrical energy and/or water availability due to emergencies such as extreme drought conditions, electric grid failure, and other severe conditions including natural and manmade disasters. The coupling between WSS and the power system was done through alternatively exchanging data between the power system and WSS simulations via a program control overlay developed in python.

A new methodology for WDS infrastructural-operational resilience (IOR) computation was developed as a part of this research to assess the real-time performance of the WDS under emergency conditions. The methodology combines operational resilience and component level infrastructural robustness to provide a comprehensive performance assessment tool.

The optimization-simulation and resilience computation methodologies developed were tested for both hypothetical and real example WDS and WSCS, with results depicting improved resilience for operations of the WSS under normal and emergency conditions.

DEDICATION

This work is dedicated to my parents Mr. Nandkumar Khatavkar and Mrs. Ujjwala Khatavkar. Their love, support and encouragement made this work possible.

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1. INTRODUCTION

1.1. Problem Statement

This research focuses on the water supply system (WSS) facet of the multi-faceted optimization and control mechanism developed for an integrated water – energy nexus system under U.S. National Science Foundation (NSF) project 029013-0010 CRISP Type 2 – Resilient cyber-enabled electric energy and water infrastructures modeling and control under extreme mega drought scenarios. A water supply system (WSS) conveys water from sources (such as lakes, rivers, dams etc.) to the treatment plants and then to users via the water distribution systems (WDS) and/or water supply canal systems (WSCS). The aim is of this research is to develop optimization - simulation methodologies for real time control of WSS considering the various interdependencies between the water distribution and electric grid system under the conditions of short and long term limited power and water availability.

Methodologies are developed as a part of this research to obtain optimal controls for pumps and flow control valves for WDS and pump and gate controls for WSCS to facilitate proper functioning during normal and emergency conditions including natural disasters, power outages etc. Figure 1-1 depicts a state of art regional WDS, consisting of a supervisory control and data acquisition system (SCADA), which exercises direct control over the entire operations of the WDS including the pump and valve controls.

The state of art in practice is to make the decisions with regards to the real-time controls of the WDS based on the data acquired from SCADA. It could be observed from Figure 1-1 that no data acquisition or transfer is currently done in between the power plants and the WSS.

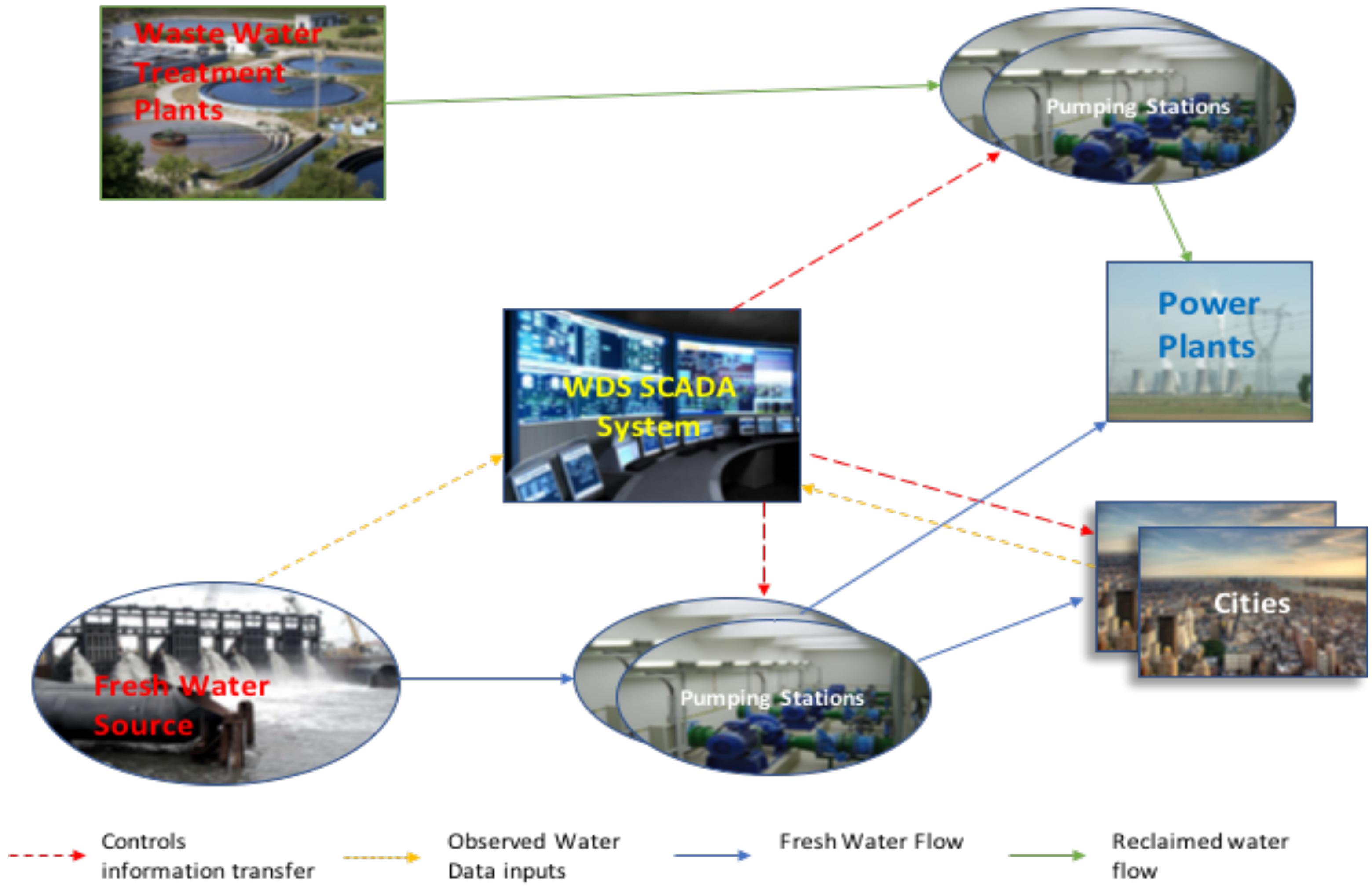


Figure 1-1 Schematic of a State of Art Regional Water Distribution System

Water and energy distribution systems have several interdependencies, which in current state of practice are not considered for the real-time control of these systems. These interdependencies include the cooling water requirements of the power plants (satisfied by the regional WSS) and the power requirements of the pumps (supplied by the regional power distribution system, PDS). Figure 1-2 gives the proposed schematic of the regional WSS with the optimization – simulation methodology being employed for real – time control of the WSS. It could be observed here that a much more comprehensive structure of information sharing, and control information transfer is achieved through application of this methodology. The optimization – simulation model communicates with the SCADA system, which itself communicates with all the major components of the system. A salient feature of this methodology is that data acquisition and valve control is now exercised at the power plants as well as the waste water treatment plant (WWTP) through the WDS and/or WSCS SCADA, therefore providing an optimal and robust controlling mechanism.

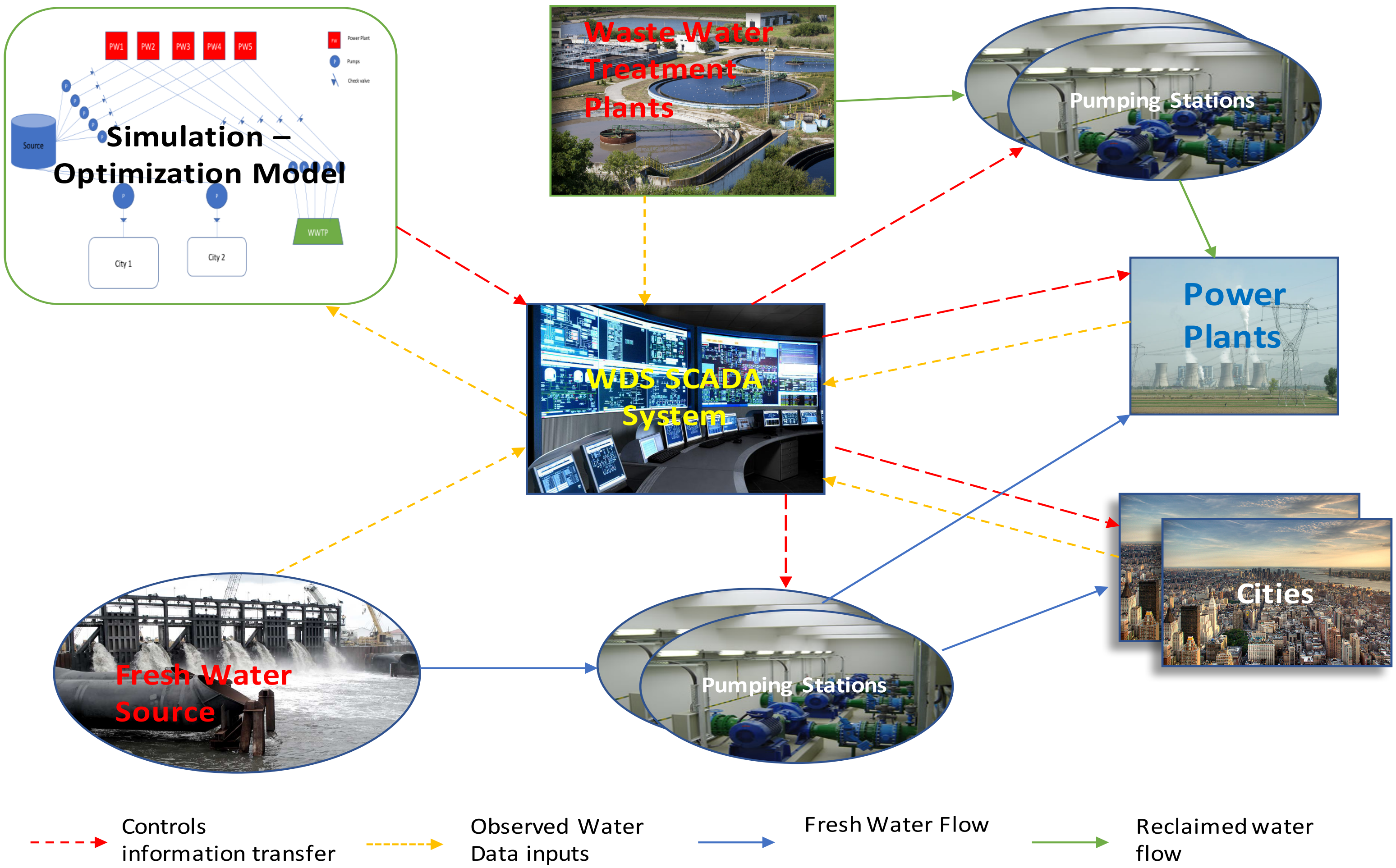


Figure 1-2 Schematic of a Regional Water Distribution System with the Proposed Methodology

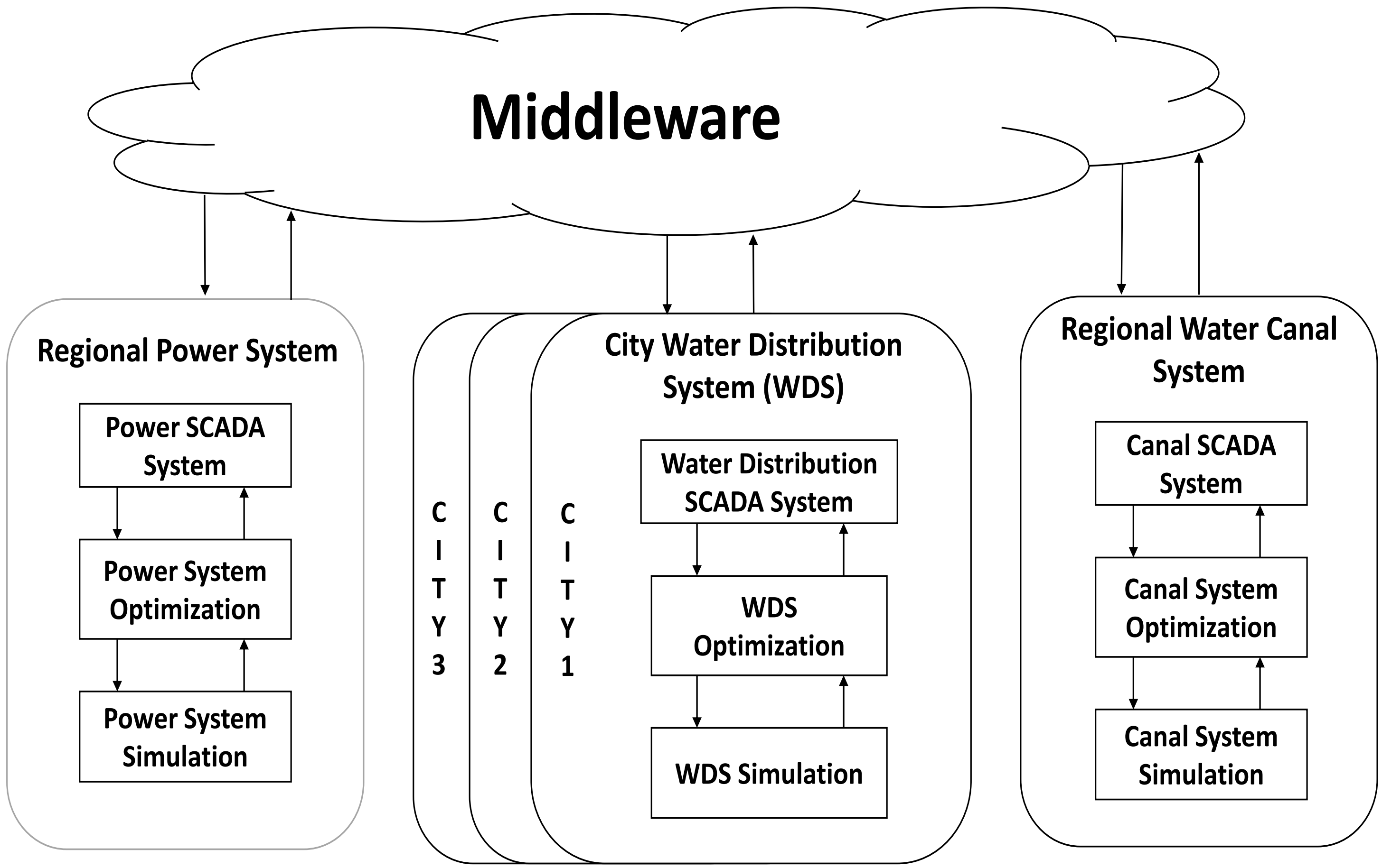


Figure 1-3 Water – Energy Nexus Combined Optimization – Simulation Methodology

1.2. Water Supply Systems (WSS)

A water supply system (WSS) consists of pressurized water distribution systems (WDS) and water supply canal systems (WSCS). Access to safe drinking water is a basic requirement of a healthy population. The goal of United Nations international drinking water supply and sanitation decade from 1981 to 1990 was safe drinking water for all (Bourne, 1981). While a considerable effort was made by the United Nations in achieving these goals, the population growth in the developing regions of the world wiped out the gains. This depicts the complexities and difficulties faced by the designers and managers of modern water infrastructures. The United Nations world water development report: Water for people water for life highlighted for the first time the effects of an inadequate supply of safe drinking water on the lives of people in the developing nations (United Nations World Water Assessment Programme, 2003). The report goes on to state that the lack of energy and infrastructure required for water supply systems limits women's productive and community development activities leading to numerous social problems.

1.2.1. Water Distribution Systems (WDS)

Water distribution systems (WDS) are intricate networks consisting of several components and processes. Figure 1-4 shows the layout of a typical water distribution system. The input for a typical water distribution system is from a raw water source which could be a surface water source or a groundwater source. This water is pumped through a raw water pumping station and thereafter undergoes a series of treatment processes for enhancement of its quality for public health requirements. Thereafter, the treated water is stored in tanks and distributed to the consumers through the WDS. A WDS performs its functions through a series of processes through its several components, subcomponents and sub-subcomponents. Figure 1-5 shows a hierarchical structure of the various components of a WDS. It could be observed that a typical WDS has three main components viz. pumping station, distribution storage and the distribution piping.

Deterioration of aging water supply infrastructure and nonexistence of a modern water supply system are two factors responsible for lack of adequate safe drinking water supply in urban areas across the globe (Mays, 2000). The reliability of a system is the probability that the system performs its intended function in an operational context for a specific period (Sarno et al., 2005). One of the major concerns today is the continually decreasing reliability of the existing aging systems (Mays et al., 1989). Apart from maintenance and replacement of water supply system components, operations of a water supply system play a major role in improving or degrading reliability of the system. Bao and Mays (1990) associate the reliability of water distribution systems with two types of failures including mechanical failure and hydraulic failure. The mechanical failure is associated with system failures due to pipe breakage, pump failure, power outages, control valve failure, etc.

Hydraulic failures include system failures due to inadequate flow and pressure heads at one or more demand points because of changes in demand and pressure head requirements and hence are closely related with operations of a water supply system. Hydraulic reliability is a measure of the performance of the water distribution system (Bao and Mays, 1990). Controls of water distribution system (WDS) include controlling the different types of valves and pumps in the system.

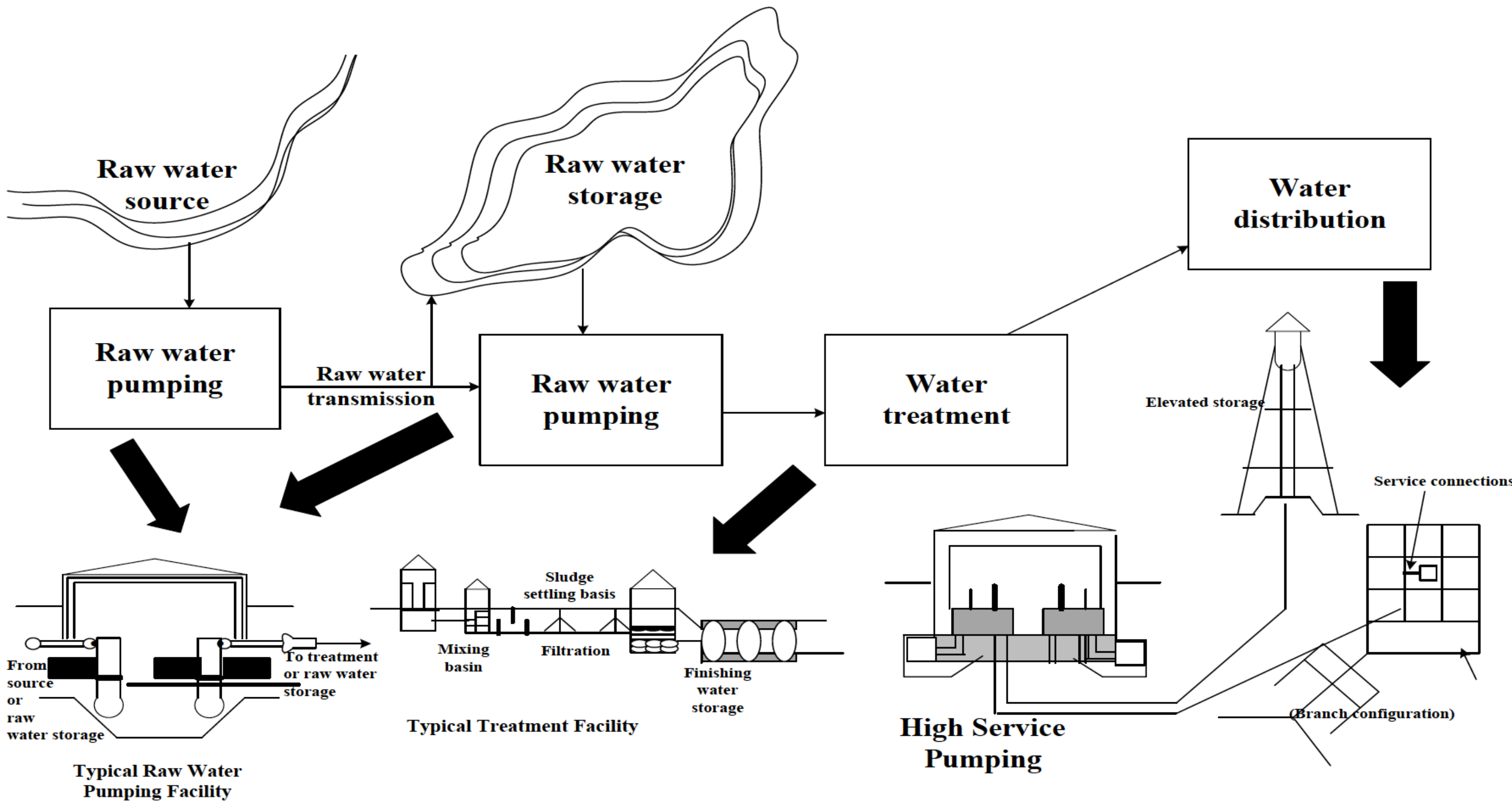


Figure 1-4 Layout of a Typical WDS (Cullinane, 1989)

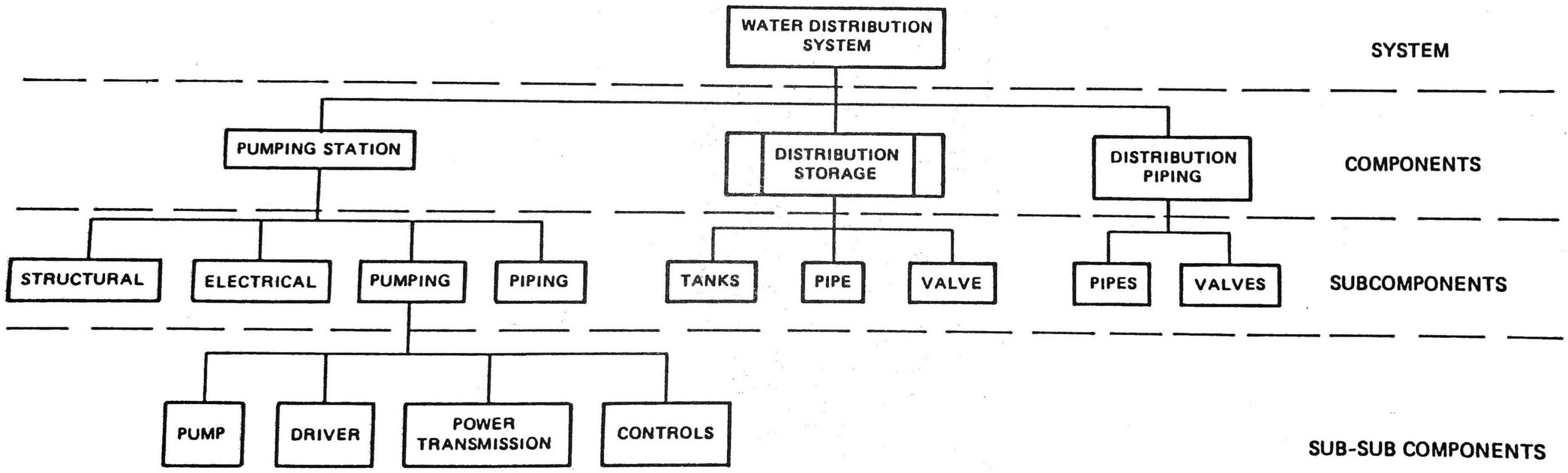


Figure 1-5 Hierarchical Relationship of Components, Subcomponents and Sub-Subcomponents for a Water Distribution System (Cullinane,1989)

The American Water Works Association (2010) presented the various principles, practices and guidelines for operation of water supply systems. The most typical type of pump used in water supply system is a centrifugal pump. Operation of pumps vary for different pumps and it is required that the specific recommendations of the manufacturer be consulted before operating any unit. Typically, centrifugal pumps used in water supply systems are operated at a constant speed, while the system pressures are controlled by using different combinations of pumps at different times by switching pumps on and off. One of the major concerns in operation of pumps by switching them on and off frequently is the excessive motor wear. Medium-sized motors should not be cycled (i.e. started and stopped) more frequently than 15 minutes and larger motors should not be cycled even less frequently (American Water Works Association, 2010). The several considerations for pump operations are discussed in detail in later chapters.

Another important component of WDS which is used for control is a valve. There are several types of valves used in WDS to exercise control over treatment processes, pumps and other equipment. Valves used in WDS are designed for different purposes including isolation of piping (isolation valves), regulating pressures (pressure control valves), controlling flows (flow control valves), preventing backflow (control valves) and relieve pressures (pressure relief valves). Valves could be also classified based on the control mechanism used such as ball valves, butterfly valves, disc valves, clapper valves, check valves, choke valves, diaphragm valves etc. Most valves installed in a WDS are intended to start and stop flows, i.e. they are designed to be either fully open or closed under normal conditions (American Water Works Association, 2010). Valves are not used for throttling flows and should not be opened or closed partially.

Operation of pumps is a major cost consideration in the operation of water distribution systems. Pump operations are optimized to minimize power usage particularly during peak hours of operation. An optimization model for pump operation costs considering uncertainties in water demands is presented in chapter 3. About 7% of the total energy consumed in the United States is used by the municipal water utilities (Brailey and Jacobs, 1980). Goldstein and Smith (2002) predict the future electricity requirements for the first half of the 21st century to be around 4% of the total energy produced in the United States, while the average energy requirement for cooling water required for all types of power plants is estimated at around 0.5% of production from such a plant. Chapters 4 and 5 explain the development and application of a WDS pump and valve control optimization – simulation model for real-time operations during limited electrical energy and/or water availability conditions.

A typical pumping plant includes several pumps working in parallel to meet the demands in the network. The plant is designed to operate at the maximum capacity to meet the maximum quantities of forecasted normal and emergency demands in the system with a certain factor of safety. Although the system is designed for a larger discharge, the demands observed on a day-to-day basis are much smaller than the maximum design discharges. In addition to the excess pumping capacity provided in the system as a factor of safety, a typical water distribution system also consists of elevated storage. It is a general practice to pump water to these storage systems during the non-peak hours of power consumption for economic reasons. This storage water is then used for supply during the peak hours making the operations of the pumps in the system time dependent. The state of art in practice currently is to prepare pump operation schedules (switch on and off times) based on the levels of water in the storage tanks. Whenever the water level in the tank falls below a certain amount, the pumps are brought into action to fill it up. This could lead to rapid on and off switching of the pumps causing excessive wear. An optimization-simulation methodology for control of pumps and valves considering tank turnover requirements is presented in chapter 6.

1.2.2. Water Supply Canal System (WSCS)

Water supply canal systems (WSCS) are used to supply water for agriculture, municipal and industrial use, fish and wildlife enhancement, decreasing flood damage and power generation (Buyalski et al., 1991). WSCS can be large consumers of energy for pumping especially at locations where the topography of the region mandates lifting of water. For example, the Central Arizona Project (CAP) is the largest single consumer of electricity in Arizona (Lamberton et al., 2010; Scott et al., 2011; Eden et al., 2011). CAP uses approximately 2.8 million megawatt-hours of energy (about 4 percent of all the energy consumed in Arizona) to deliver about 1.5 million acre-feet of water to central and southern Arizona (Lamberton et al., 2010). Pumping plants lift water 2900 feet over the length of the CAP canal. Scott et al. (2011) documented in detail the water-energy nexus policy implications for canal systems in general and CAP in particular. Cost of energy required for pumping is about 80% of the total cost incurred for urban water supply in the United States (Lamberton et al., 2010). The reliability of canal operations during natural and manmade emergency conditions is an important factor while considering the interdependencies between the water and energy systems. An optimization-simulation methodology for real-time control of WSCS was developed as a part of this research and is presented in detail in chapter 8

Modern water supply canal systems (WSCS) are equipped with a SCADA system similar to a WDS. WSCS SCADA system enable automated control of canal gates and pumps and acquire real-time canal data including water levels in the canal, flow velocity and water quality data. Figure 1-6 shows typical SCADA enabled WSCS gates used for canal supplying raw water to city of Tempe, AZ. Figure 1-7 shows a raw water storage facility for Tempe, supplied by the canal.

Figure 1-6 SCADA Enabled WSCS Gates Tempe, AZ

Figure 1-7 Raw Water Storage Facility, Tempe, AZ

1.3. Paradigms of WSS Design and Operations

Mays (2000, p.1.1 – 1.2) gives a brief description of the history of development of water distribution systems. The author states that the first efforts to control the flow of water were made in Mesopotamia and Egypt, where remains of prehistoric irrigation canals still exist. Mays (2000) gives details of water distribution systems developed by several ancient civilizations like Knossos, Crete, Minoan, Acropolis, Anatolia etc. Several incredible structures including pipes, canals, tunnels, inverted siphons, aqueducts, reservoirs, cisterns and dams are observed at these ancient sites. Mays (2010, p. 95 - 100) gives a detailed description of the design methodologies employed at the various ancient civilizations. Though the design methodologies evolved to an incredible degree of sophistication from simple cisterns of the Neolithic ages seen in various Greek civilizations to the famous aqueducts of the Roman empire, it could be seen that the fundamental principles and understanding of water systems remained uniform throughout the globe during this period. This ancient 'understanding' of water systems could be termed as the first 'paradigm' as defined by Kuhn (1962, p. x), as this understanding was universally accepted and provided sufficient solution for the water distribution to the various civilizations of that era. The term 'sufficient solution' could be used to emphasize on the fact that human expectations of the water distribution systems have undergone considerable changes since the advent of the first paradigm, therefore, the solution provided by any water management paradigm would be just an adequate or sufficient solution for the popular expectations of that period rather than a perfect one. Kuhn (1962, p.18) gives Francis Bacon's acute methodological dictum as 'Truth emerges more readily from error than from confusion'. This principle was realized greatly by ancient water system designers, since their designs were based on observations and experiences rather than any scientific theories and hence were based on trial and error methods. The trial and error methods employed here were completely based on the designer's faith in the various observations and experiences of past. The first paradigm was completely based on gravity flow and hence, the designs were quite simple compared to most of the modern water distribution systems. There was a fundamental limitation in this system, wherein the water could just flow downstream, and water could not be lifted through a height. The design process for water systems in this era was completely based on faith of the designer in his observations and experiences, rather than firm scientific theories.

Water distribution systems function as conveyance system for water from the source to the treatment facilities and thereafter to the individual homes through a series of pumps, pipes, tanks and other accessories (Garg, 2010, p. 25). In ancient civilizations, this system was quite simplistic with a few components and the source being comparatively nearby but as the civilizations flourished and developed, it became a challenge to supply safe drinking water to the populace. Many new cities that flourished during the beginning of the industrial age lacked a functional water distribution system. Localized sources of water such as wells, ponds, lakes etc. were used as primary sources of drinking water for the society. But in the absence of a well-functioning sewerage system, these sources were highly polluted and posed a health hazard for the community. Epidemics and mass poisoning was a regular occurrence in the medieval industrial communities. Hence, a need was felt to provide for a safe water supply system for the industrial towns and development and research in the field was the instigated thus.

A real tension between science and faith in the water management field started only at the advent of industrial revolution in mid – 19th century Europe. Industrial revolution has multi-faceted effects on the global society. As far as water systems are considered, industrial revolution was single handedly responsible for a major shift in water technologies as well as management methodologies from gravity canals to much more sophisticated and scientific systems including pressurized flow using pumps and piping systems. This paradigm changes also led to inclusion of basic water treatment in urban water infrastructures leading to a change in the perspective of water from just a commodity demand in quantities to a multi-faceted demand of quality and quantity for different applications. After the discovery of water-borne diseases and water being identified as a conveyor of disease causing agents by Dr. John Snow in latter half of 19th century, water management became more of a health issue rather than just an engineering challenge. This opened a new can of worms as far as crisis are involved, and there was a time in Great Britain, when beer was seen as safer of a bet than the water being supplied to the citizens. This led to what we can term a 'Water supply as health measure' paradigm in the water management chronology. For most of the 20th century, water systems across the globe were designed based on these two paradigms viz. 'demand quantification paradigm' and 'Health safety paradigm', wherein the only functions the water distribution systems performed included demand satisfaction of the people as per the water rights and maintenance of water quality at the consumer end to avoid breakouts of epidemics of diseases like Cholera. Mays (2000, p. 1.9) gives a detailed description of the state of water distribution systems at the end of 19th century. This era depicted the prevalence of science over faith in the sense of replacement of methodologies based on faith in the form of experiences and observations by more reasonable and scientific procedures.

As water treatment and distribution facilities became common in most of the cities in the industrialized world during the early years of the 20th century, various scientific methods were developed to design these systems. Because of rapid urban population growth, it has become a challenge to meet the resulting water demands and hence, is necessary to design intricate water distribution systems to meet the design criteria and gain better control on the water distribution. With the design techniques developed, there was also a need to ‘test’ or simulate these designs before implementation of the same. Various physical models were used for some decades before development of the sophisticated computer models available today. A detailed explanation of the design techniques for distribution systems and the development of various models, is given by Mays (2000, 2002).

Over the last decade, the theory employed for management of water across the globe has undergone revolutionary changes as defined by Kuhn (1962, p. 166). Kuhn (1962, p. 66) states that a revolution has materialized only because of scientific discoveries or lead to a certain scientific discovery but it does not start without a crisis. As discussed earlier, Industrial revolution led to several crises including those related to water rights and health of the general population in water distribution systems design, that led to a revolutionary change in the understanding of water systems and led to development of first ‘scientific paradigm’ of water management. The field of water management is once again believed to be on the verge of revolution because of the various crises developing because of crises of governance, increasing uncertainties due to global climate change leading to reduction in predictability of the boundary conditions under which the water systems were earlier planned and designed, the need for sustainable water management including water recycling and reduce and the acute scarcity of water required to provide for the ever increasing populations across the globe.

Pahl - Wostl et al. (2006) give a description for changes in water management paradigms throughout the last decade. A table giving the old and emerging paradigms could be found in this newsletter. This newsletter though giving an excellent description of paradigm shifts in the field of water management policies, it fails to provide an insight into the subsequent paradigm shifts occurring in the technologies as well as the technical operation methodologies employed for management of water distribution systems. Figure 1-8 highlights the various paradigm shifts given by Pahl - Wostl et al. (2006) which occurred in the field of water management over the last two decades.

Daigger (2009) asserts that 'the 'linear' approach currently used including take, make and waste approach when applied broadly to natural resource use, is becoming increasingly unsustainable'. A toolkit which reduces the net urban water abstraction from the environment, thereby relieving urban water stress and reducing resource consumption and nutrient dispersal is proposed in this work. The toolkit includes stormwater management/rainwater harvesting, water conservation, water reclamation and reuse, energy management, nutrient recovery and source separation. The toolkit described here meets all the conditions described by Kuhn (1962) for development of a new paradigm. The methodology is universally accepted, developed to address a crisis and provides for a change in the fundamentals of the state of art in practice. Thus, the toolkit described by Daigger (2009), could be considered as a new paradigm, which might be referred to as ‘sustainability paradigm’ in the water management field. Decentralization is one of the major themes of these paradigms, which provides for a network of local smaller interconnected water distribution systems rather than one large system for the entire area. The major advantage reported for the case of decentralization includes better control over a smaller jurisdiction rather than a larger one.

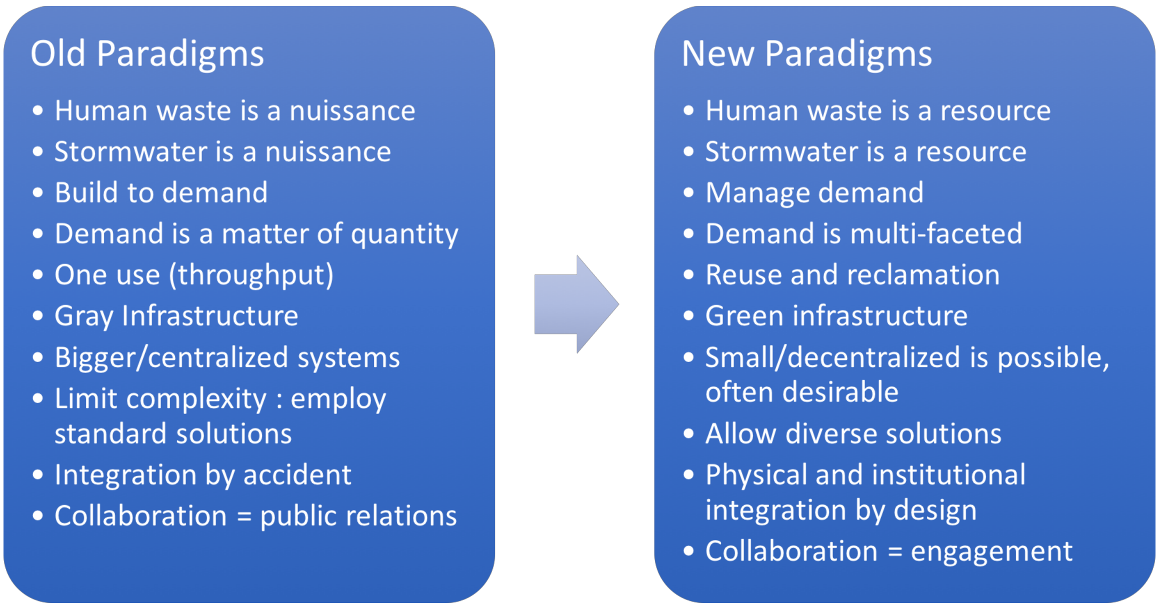


Figure 1-8 Highlights of Paradigm Changes (Pahl- Wostl et al., 2006)

1.4. Resilient Water Infrastructure

Resilience is one of the most discussed terms in the engineering discipline during the last several decades, which has led to development of several paradigms and definitions leading to a widespread confusion. One of the most widely discussed definitions of resilience is ‘the ability of a system absorb disturbance and reorganize to retain essentially the same function, structure, and feedbacks – to have the same identity.’ (Buckle, 2005; Walker and Salt, 2012 and Walker et al., 2016). This could be termed as the ‘robustness’ approach, since this definition deals with the characteristic of the system to cope up with predetermined and predicted changes that occur from time to time. This definition fails to address the resilience that the system displays during regular course of operations and rather deals with just the contingencies in the system performance.

Another definition of resilience that is well documented is "resilience is a measure of robustness and buffering capacity of the system to changing conditions" (Agudelo Vera et al., 2012; Berkes and Folke, 2006). This definition relies on 'buffering capacity' to broaden the concept of resilience that includes redundancy for the various components of the system. The deficiency of this definition fails to provide a comprehension of the normal operating conditions of the system as well as the non-functional redundancy added by the 'buffering capacity'.

Water infrastructure plays an important role in the survival and sustenance of any modern society. The task of construction, operation and maintenance of water infrastructure is a responsibility of the respective water agencies and administration. The basic function of the water utilities is to obtain water from a source, treat the water to an acceptable quality and deliver the desired quantity of water to the appropriate place at the appropriate time. Mays (2002) presented a detailed description of various components and features of the water distribution infrastructure. Clements et al. (2010) broadly define water infrastructure as 'the basic physical and organizational water-related structures needed for the functional operation of society. These include both built (e.g., reservoirs and retention systems, piped collection and distribution systems, treatment systems) and natural infrastructure (e.g., forested land, stream buffers, flood plains and hydrologic networks, wetlands).'

There is no doubt that a proper design of the built part is imperative for undeterred functioning of the infrastructure in serving the consumers, but another important aspect for sustainability of this infrastructure is a good strategy for real-time operations. The natural part of the infrastructure is controlled and regulated through various built structures such as canals, dams etc. Real-time operations of a water distribution infrastructure include controls and operations of both the built as well as natural infrastructure associated with water distribution. Che (2015) developed a model for optimization of real-time controls for dams which form an important infrastructure controlling the natural part of the overall water distribution network infrastructure. He employed a combined methodology of hydrologic and hydraulic modeling, short term rainfall forecasting and optimization and reservoir operation models. Though water infrastructure is one of the critical infrastructures for a city, lack of maintenance and non-optimal operations are often observed in this regard even in industrialized countries. In United States, most of the water infrastructure was constructed in the mid – 20th century with a lifespan of 75 – 100 years. Even with a decrease in water consumption associated with losses over the years, there are still an estimated of 240000 water main breaks per year in the United States, wasting over 2 trillion gallons of treated drinking water (ASCE, 2017). ASCE (2017) emphasized in the report that due to the critical need of the water infrastructure, significant new investment and increased efficiency in operation and maintenance is needed for the various components of the urban water infrastructure as filtration plants, pumps and pipes across the nation age past their useful life. The report estimates that leaky pipes waste about 14 – 18% of treated drinking water, which could serve about 15 million houses every day across the country and the situation is expected to worsen in the coming years. This shows the urgency of drastic change in the policies associated with operation and maintenance of the water distribution infrastructure across the United States. Resilience with respect to operations and design of such infrastructure is of national importance and hence it is utterly necessary that sound policies are formulated in this regard.

Vulnerabilities of a system are rightly defined as the internal features or externalities directly or indirectly affecting the performance of the system in achieving its valued functions. A water distribution system like most of the other public infrastructures is plagued with several vulnerabilities. Major natural vulnerabilities of the water infrastructure include climate change, prolonged droughts, natural disasters, etc. Several human induced vulnerabilities exist with respect to the capacity of the water infrastructure to provide water to the populace including but not limited to population growth, water pollution, wars, water treaties, human – induced climate change, etc. Prasad (2009) highlighted the various vulnerabilities affecting urban infrastructure and discussed generalized policy solutions for the same. The paper highlights more into the policy issues with regards to disasters affecting the smooth functioning of a water distribution system among other water infrastructures.

An important consideration for sustainability of water distribution systems is climate change. Piratla (2012 and 2016) considered sustainability of water distribution networks with regards to their impacts on the environment while considering the operational reliability and sustainability of the system. Several things associated with water distribution infrastructure could be detrimental to the environment. Construction of large dams, could lead to displacement of several people and destruction of biodiversity at large. Otto-Zimmermann (2012) discussed the implications and effects of urban infrastructures on climate change and vice a versa. The book emphasizes on the growing recognition of the profound effects of climate change on urban infrastructure across the globe. One of the worst affected infrastructures is expected to be water distribution systems, resulting from their dependence on the hydrologic and meteorological cycles of the planet's atmosphere. Prasad (2009) investigated techniques for improvement of sustainability and resilience of cities with regards to the climate to make the infrastructure robust enough to withstand the foreseen effects of climate change.

To ascertain reliable and resilient operations of the overall water distribution infrastructure, an important task is to forecast and predict the various contingencies which may occur in near as well as distant future. It is also important to consider a reasonable level of uncertainty in the various predicted contingencies. Scott et al. (2012) explained the scenario planning involved in addressing critical uncertainties in water distribution system operations under conditions of water scarcity and rapid development. Water scarcity could be due to sudden increase in population, deficient planning, geographical and topographic scarcity, short term or long-term droughts, political reasons etc. Prediction methodologies for water availability are an important consideration for planning the operation policies of water systems. The study presented here deals with developing a methodology for optimal controls of urban water distribution systems to ascertain resilience and reliability of the overall system in fulfilling their valued functions, which include provision of water to the appropriate place and time while satisfying the hydraulic and quality requirements. Chapter 7 highlights the resilience computation methodologies developed for real-time operations of WDS under emergency conditions.

1.5. Water – Energy Nexus

Critical infrastructures are complex physical and cyber-based systems that form the lifeline of modern society, and their reliable and secure operation is of paramount importance to national security and economic vitality. Among the critical infrastructure systems, the electric grid and the water treatment and delivery system are highly automated in terms of their operation and control, each of which is facilitated by a cyber-based supervisory control and data acquisition (SCADA) system. The SCADA systems for these two interdependent physical systems operate independently and do not share any information or common observability/control capabilities. Disruptions, either natural or man-made, in any one of these systems can adversely impact the other because of the inherent system interdependency. As an example, in July 1993, the Raccoon River in Des Moines, IA, flooded several electrical substations belonging to Mid-American Energy, because of which there was widespread disruption of electric supply to the downtown Des Moines area and surrounding neighbourhoods. The primary water treatment and delivery facility for the city of Des Moines was severely flooded due to the inability of the disabled drainage pumps to keep water away from the facility. Excessive flooding led to serious contamination of the water treatment equipment affecting the water supply to several localities in Des Moines for over a month and resulting in economic losses and hardship to the residents in these localities.

Water follows a series of stages in an urban water supply system starting with drinking water processes and followed by customer use and finally the wastewater treatment stage. The drinking water processes include all such activities involved in conversion of raw – water to water safe for human consumption including extraction of raw – water, water treatment and distribution of treated water to consumers. The customer use stage of water life cycle includes use by residential/commercial and industrial customers. The last stage in the water life cycle consists of various processes such as wastewater treatment, effluent discharge and reuse etc. Every stage involved in urban water life cycle is energy dependent and involves a high level of power usage. The largest energy consumption is seen in case of the water distribution stage, which may account for 80 to 90 percent of the energy used to supply drinking water in some systems (United States Government Accountability Office, 2012a). United States Government Accountability Office (2011) gives the various factors to be considered while formulating the national policies for water – energy nexus operations including (1) varying local impacts of federal energy and water policy choices, (2) the mitigation of barriers to using innovative technologies and approaches, (3) the challenge of making effective policy choices in the absence of more comprehensive data and research, (4) the importance of coordination among governmental and non-governmental stakeholders to improve planning, (5) the attention to the uncertainties that affect energy and water resources when setting and implementing federal policies for these resources.

The interdependencies of a water distribution infrastructure over the electric grid depend upon several factors regarding the layout and components of the system. Figure 1-9 summarizes the interdependencies between the two systems. Major energy consumption involved in conveyance of water from the source to treatment facility and finally to the customer is due to pumping. The amount of energy required to convey water from the source to the treatment facility is considerably higher in case of a groundwater source as compared to a surface water source, since a system fed by a surface water source could make use of gravity for water conveyance over large distances. In addition to the type of raw-water source, the topography and location of the pump affects the energy consumption. The United States Government Accountability Office. (2011) highlighted the effects of topography on energy consumption for water distribution systems by considering the example of San Diego, CA. San Diego's water requirements are quenched through water from northern California, transporting its water through hundreds of miles and lifting it about 2000 ft in the Tehachapi mountains. This transportation and lifting of water is highly energy consumptive and shows how energy and water systems are interdependent on each other.

Another important factor affecting the energy use in a water system includes the quality of water being treated. Good quality raw-water requires minimum treatment and hence the energy required for water treatment is reduced considerably. Condition and age of water distribution system largely affects the capacity of the water system to supply water with minimum energy consumption, since older the system, lesser is its efficiency of its components. Recently the American Society of Civil Engineers (ASCE) categorized the American water infrastructure with a 'D+' grade, which shows that the age and the condition of the systems is of utmost concern. Rice (2011) highlighted the need of optimising the water systems for electricity consumption including selection of water sources considering the topography and location. Schnoor (2011) and the United States

Government Accountability Office. (2012 b) provide compelling evidence of interdependencies between the water supply system and energy sources other than electricity with a concentration on the newly developed fracking technology for extraction of natural gas from great depths of earth.

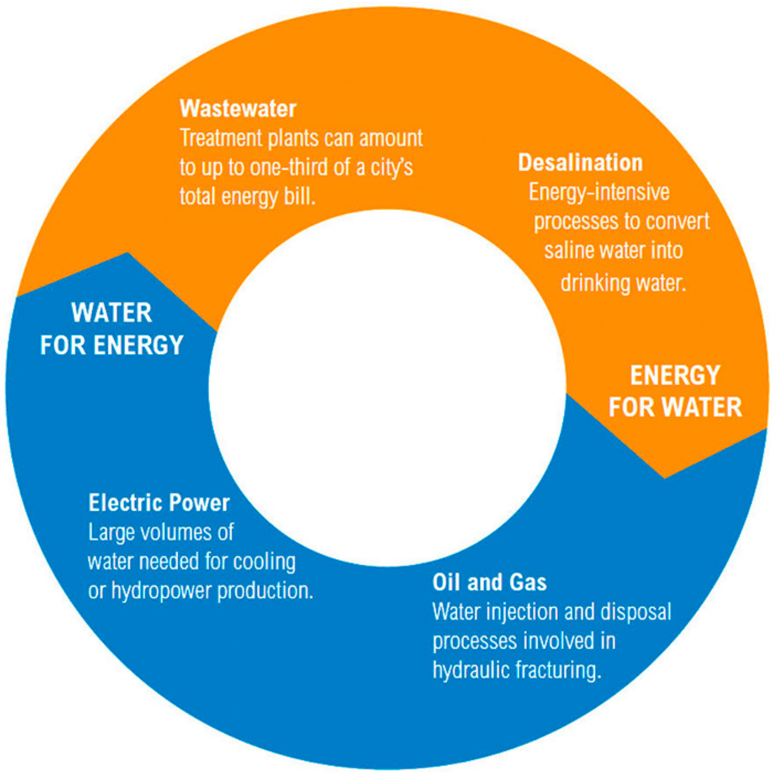


Figure 1-9 Water – Energy Nexus (Source: http://www.pumpsandsystems.com/pumps/may-2016-water-energy-nexus-business-risks-rewards)

This shows that almost all the energy systems across the globe are closely interdependent on the local water supply systems and vice a versa, thus forming a combined system termed as the water – energy nexus. This project would concentrate more at the water – electricity nexus rather than water – energy nexus and hence the term 'water – energy' nexus has been used here with a concentration on the electrical grid system rather than the entire domain of energy.

Similar interdependencies also exist between the water supply system and the electric grid in terms of cooling water for generating plants. This proposal aims to develop a model of the interdependent cyber-physical systems. A key contribution of the proposed work is to utilize the developed model to build resiliency in the physical infrastructures to extreme operating scenarios and natural disasters. As such, the model will be specifically used to analyse extreme scenarios of mega droughts predicted in the U.S. Southwest by NASA, in addition to other significant disruptions in each of the infrastructure systems to examine the impacts and develop effective mitigation measures. Various scenarios of contingencies (faults or disturbances) and extreme conditions will be envisioned and a behavioural analysis of consumer usage through carefully designed survey experiments conducted for water and electricity consumers to determine how such disruptions or limited availability would affect their use of each of the commodities. A sophisticated consumer usage model under extreme conditions would then be developed. This information would be fed into the mathematical model for the interdependent infrastructures to examine the impact on the systems.

Carter (2013) provided several figures regarding the energy sector's water use in US. Committee on Energy and Natural Resources, United States, Senate, and Congress. (2014) provide information on the state of art in practice for management of the water – energy nexus. This white paper provides compelling evidence and grounds for bringing about a change in the current state of art for water – energy nexus, which are treated as separate entities rather than interdependent ones across the US. The paper states that by

2040, US electricity consumption will increase by 28 percent, natural gas production will increase by about 67 percent, and oil production will increase 32 percent from the 2011 levels. These figures are in accordance to a predicted 22 percent rise in US population. The paper estimates roughly around 11% of the water withdrawn in US is currently used for energy – related purposes including cooling water for power – plants and water required for fracking and oil extraction. One of the points the paper stresses upon is that about 41% of the water withdrawn from the water – cycle is used for thermoelectric cooling, which accounts for about 6% of the total US water use. Considering these figures, it is obvious that there is an urgent need in optimization and better control mechanisms for the overall water – energy nexus to achieve sustainable and reliable operations in the future.

Water – energy nexus could be viewed from both, a national point of view as well as a more local or a regional point of view. Howells and Rogner (2014) explain the various methods of integrating the water and energy systems both at regional and local level. A congressional committee report on the water energy nexus demonstrated that a scheme of regional level control over the operations of the water – energy nexus could be more efficient as compared to several local control mechanisms. This is also evident from the design of the two systems, since water systems usually cover quite smaller areas than an electric grid because the water system's design and operational dependencies on the location and topography in general. Healy et al. (2015) presented a study of the water – energy nexus through an earth – sciences approach, wherein the authors explain the interdependencies and complexities of the two systems and the uncertainties thereof. It is stressed upon in this study that an integrated mechanism of control for the two systems could be a better management practice that the present scenario.

1.6. Phases of Research

The research presented herein is aimed at the development of an overall control methodology for real-time operations of the water – energy nexus. The two systems being independently operated in state of art in practice, have several interdependencies and the methodology proposed here involves using these interdependencies to improve the reliability of the nexus operations under emergency conditions of extreme droughts or power outages because natural or manmade disasters. The proposed research is performed in a sequence of logical phases, summarized in this chapter.

The overall research was performed in several phases starting from review of state of art real-time control methodologies for the water – energy nexus. Figure 1-10 gives an overall perspective of the research work undertaken and proposed for the future. The project work was initiated with the conceptualization phases (phase 1, 2 and 3). These involved reviews of the previous work done by researchers on operation of water distribution systems, which provided this project with a firm ground and a realistic scope. Thereafter, the process involved development of various optimization models (phases 4, 5) for real-time control of the WDS. The optimization models mathematically formulated in these phases form the backbone of the overall effort.

After mathematical formulation of the optimization models, the next step was development of computer codes for application of the overall methodology for real-time control of WDS. Phases 6, 7, 8 and 9 involve development of computer codes and application of the methodology for several scenarios for realistic example WDS's and power distribution systems (PDS). With the basic methodology developed and applied for an example WDS, several modifications could be performed to obtain a more comprehensive methodology for developing a more comprehensive real-time control system for the WDS and the water – energy nexus. Two important improvements to the basic methodology include consideration of water quality and water age constraints in the optimization models developed in phase 4 and 5 and developing a strategy for improvement of reliability, resilience and robustness of the system operations. Phases 10 and 11 deal with these improvements to include the water quality and reliability, resilience and robustness considerations in the models. Phases 12 and 13 involve the last part of the project which includes formulation of contingency plans, operation procedures, strategies and development of a methodology for canal controls for a regional WDS. The last part of the research would facilitate instrumentation and application of the proposed methodology for a WDS operator.

1.6.1. Phase 1: Literature Review

A detailed literature review was performed with the motive of knowing about similar previous research performed by various researchers around the globe. The pros and cons of the previous research literature provided a basis for formulation of the next stages of this research. The various topics reviewed included historic overview of water distribution systems (WDS) computer simulation, development of water distribution systems optimization methodologies, literature review for water – energy interdependencies, resilient infrastructure, water distribution systems reliability, risk management and reliability studies for water – energy combined system and evolutionary methods of optimization.

Figure 1-10 Overall Research Efforts

1.6.2. <u>Phase 2: Identifying the Purpose and Scope of the Research to be Undertaken</u>

After the exhaustive literature review, a study was performed to identify the various problems encountered in the operations of a water distribution system as well as the effects of a cascading failure of the water distribution and electrical systems. This phase included study of the various water and power facilities and their interdependencies. Though the various basic components and operating principles are universal, there are several design and topographic conditions which change from one system to another. Some water distribution systems are designed for an intermittent flow while most of them are designed for a continuous supply. Also, the system might consist of several demand and pressure zones having different demand and head requirements. These among several others are the real-world boundaries which are to be considered while applying the real-time control methodology for any WDS. Another important real-world boundary for the water – energy nexus is that there is no central control or explicit control over the entire system, but a network of several control points, which are to be managed implicitly through a middleware.

1.6.3. Phase 3: Formulation of Overall Model for Operation of Water Systems under Contingencies

This phase involved mathematical formulation of an unconstrained optimization model for determining pump operations for normal as well as emergency periods. Both constrained and unconstrained optimization models were formulated as a part of this phase. This phase included research and mathematical formulation of the problem related to operation of water systems under various contingencies such as component (such as pipe, pumps, valves etc.) failure, demands exceeding capacity, failure to supply quality water at the user end etc. The mathematical models developed as a part of this phase included a pump scheduler, an optimization model considering pressure head, supply – demand constraints and an interface with a hydraulic simulator (EPANET). The development of computer codes for the interface were started in this phase. Several ways were investigated to develop the overall computer code for the methodology. The modeling effort undertaken here is for the basic objective of optimizing the operations of an urban water distribution system while meeting the required levels of demand at various locations across the system. For this purpose, the objective statement formulated here considers a particular demand $D_s(i,t)$ required at a particular node 'i' at a particular time 't'. This is the demand required to be fulfilled by the system while operating optimally. Base demand at a node is an input for any hydraulic simulation model and such a model (EPANET) always fulfils all such demands across the system modelled. To optimize the system, it is necessary to perform numerous iterations through the simulator to reach a value of discharge for various nodes, such that the demands are satisfied with regards to the flow and the pressures at various nodes. In addition to these, there are numerous other constraints, which are considered as a part of the 'penalty functions' in the objective function.

1.6.4. Phase 4: Development of Model for Optimal Operation of Water Distribution Pumps with Uncertain Demand Patterns

Water distribution systems have several uncertainties associated with design, operating conditions etc. One of the major uncertainties, that a water distribution system is supposed to deal with is related to the demand patterns. Generally, WDS is designed for a certain demand pattern based on historical observations with a certain factor of safety, which adds to the redundancy rather than reliability or resilience of the system. An optimization methodology to improve the resilience of the system operations for such uncertainties was developed in this phase.

Phase 4a: Mathematical formulation for model for optimal operation of water distribution pumps with uncertain demand patterns

The model was developed as a mixed integer nonlinear programming (MINLP) problem and solved using AMPL considering both certain demand patterns and uncertain demand patterns. An optimization model for pump operation based upon minimizing operation maintenance costs of pumps for a specified demand (load) curve is presented. The purpose of this model is to determine pump operation to meet the known consumer demands as well as to satisfy the pressure requirements in the water distribution system. In addition, constraints on the number of pump ('on-off') switches are included as a surrogate to minimizing the maintenance costs. This model is a chance constrained mixed integer nonlinear programming (MINLP) problem considering the uncertainty in demand. The demand constraint is formulated as a chance-constrained problem in this model. The optimization model was solved using the LocalSolver option in A Mathematical Programming Language (AMPL).

Various optimization methodologies including non-linear programming (NLP), mixed Integer non-linear programming (MINLP), genetic algorithm (GA), simulated annealing (SA) etc. were considered as a solution methodology for solving the problems.

Phase 4b: Application of model for a realistic system

The model developed in phase 4(a) was applied to a hypothetical pumping station. A realistic hypothetical pump schedule based on the state of art in practice was used for the application. The model inputs for the example application are described here. The price of power (P_t) throughout a day in $/kw-hr was used based on realistic values. The pumping station considered in this study consists of 10 pumps with three different types of pumps. A range of maximum switches allowed was used for the implementation. Total demands are met on an hourly basis. A Q_{prod} of 55,000 gpm was used as the base demand.

The optimization model with the chance-constraint on meeting demand was applied for a range of demand satisfaction uncertainties. A decrease in the operation costs was observed with an increased uncertainty in demand satisfaction, which shows that the model further optimizes the operations considering the relaxed constraints. Model application could be extended to operations of pumping systems during emergencies and contingencies such as droughts, component failures etc.

1.6.5. Phase 5: Development of Model for Optimal Real-time Control of Water Distribution Systems under Limited Power Availability

The model was formulated an unconstrained mixed integer non-linear programming (MINLP) problem. This optimization/simulation model interfaced a genetic algorithm (GA) in MATLAB with the EPANET model within the MATLAB environment.

Phase 5a: Model for real-time operation of water distribution system considering only demand patterns

This model was developed for optimal operation of WDS to satisfy a demand pattern. The satisfied demand (D_{sat}) is the quantity of water that could be made available at a node in the system to optimize the operations of the system considering the hydraulic requirements, while the required demand (D_{req}) is the demand required at a node in the system. Required demand (D_{req}) is an input for the genetic algorithm along with the other EPANET parameters. The genetic algorithm (GA) presented here the searches over a sample range supplied by the user for computing the satisfied demands. The range could lie within a certain percentage under and above the required demands. These act as the lower bounds and the upper bounds for the D_{sat} values for the GA. The of GA, while searching for the most D_{sat} values for optimized operations of the system, also takes into consideration the various constraints related to the nodal pressures in the system. The hydraulic constraints such as tank levels, flow balance etc, are considered in the hydraulic simulation (EPANET) and thus are not required to be considered in the GA.

Phase 5(b): Model for real-time operation of water distribution system considering only pump controls to minimize energy costs

A genetic algorithm is developed to optimize the operations of a pumping system for a municipal water distribution system. The genetic algorithm is developed to solve a multiple-choice integer-programming problem in combination with a hydraulic solver (EPANET) with an aim of minimizing the pump operation costs while meeting the water demands and the hydraulic (flow) conditions of the system. An unconstrained objective function is formulated by using penalty functions for the pressure heads in the system. Other constraints related to hydraulic conditions such as flow balance, tank levels etc. are considered through a hydraulic simulation in the program EPANET for every iteration of the genetic algorithm. A value of the fitness function is thus computed for every iteration through an intricate system of data exchange between the genetic algorithm and EPANET simulation package. The genetic algorithm generates a new pump schedule for every iteration based on an initial solution provided by the user and an inbuilt crossover function, which is then passed to EPANET as an input for the next iteration. Thereafter, a hydraulic analysis is performed by the EPANET simulation package.

Phase 5c: Model for real-time operation of water distribution system considering both pump controls and demand satisfaction

A genetic algorithm has been developed here for multiple purposes of optimization of pump operations (pump schedules) and the water demand satisfaction in case of municipal water distribution system. Pumps play an important role in functioning of a water distribution system including meeting the water demands at various nodes in the system as well as maintaining pressures within a lower and higher limit required for public health and system hydraulic requirements. This makes pumps one of the major controls in a water distribution system. Another parameter being optimized here is the demands that a pump schedule can satisfy at the different nodes in the system, while meeting the pressure and flow requirements in the system. A sensitivity analysis was performed to set the various parameters for the genetic algorithm, including the weights for the different objectives in the model. An example hypothetical system consisting of residential and power plant demand nodes were used for the sensitivity analysis. This model could be used in conjunction of a power distribution optimization model as a decision-making tool for contingencies (failures) occurring with regards to the interdependent components in either system. Figure 1-11 is a flow diagram for application of the proposed models.

1.6.6. Phase 6: Research on Possible Solution Methodologies

Several solution methodologies including linear programming, non-linear programming, mixed integer non-linear programming techniques etc. were studied and considered as a solution methodology for the optimization models developed in phases 4 and 5. The complexities of the model including binary variables, iterations between the simulation and optimization models and the vast number and types of variables warranted a use of heuristic techniques. The model formulated in phase 4, involving an MINLP formulation was solved using a simulated annealing approach with an artificial neural network type solution formulator within the framework of AMPL. The models developed in phase 5 is based on the interface between the simulation model (EPANET) and the genetic algorithm in MATLAB environment. Figure 1-12 is a schematic for the optimization – simulation interface developed for the optimization – simulation methodology.

Phase 6(a): Research on solution methods for the optimization/simulation model. KNITRO, simulated annealing (SA) etc.

Phase 6(b): Computer coding MATLAB – EPANET interface and Genetic Algorithm for application of optimization models.

Phase 6(c): Sensitivity analysis and performance review for the simulation-optimization models developed

This phase included an exhaustive sensitivity analysis for determining the penalty weights for the various models developed in phases 5(a), 5(b) and 5(c). The sensitivity analyses performed included those for determining genetic algorithm parameters such as the population size and generation limit (function calls), stall generations number, fitness function etc. and the various weights associated with the penalties within the fitness function.

For a successful implementation of a genetic algorithm for solving an optimization problem, numerous inputs in form of parameters are required. These include population size, generation limits, fitness limits, etc. Since a genetic algorithm is a heuristic solution methodology (it is a search technique which searches for solutions over a sample space iteratively), it is requires various stopping conditions and bounds to avoid an infinite search. These conditions and parameters vary with the model being solved and the number of variables solved in the system. A sensitivity analysis is thus required for deriving the various parameters and bounds most suitable and giving satisfactory results for an optimization model.

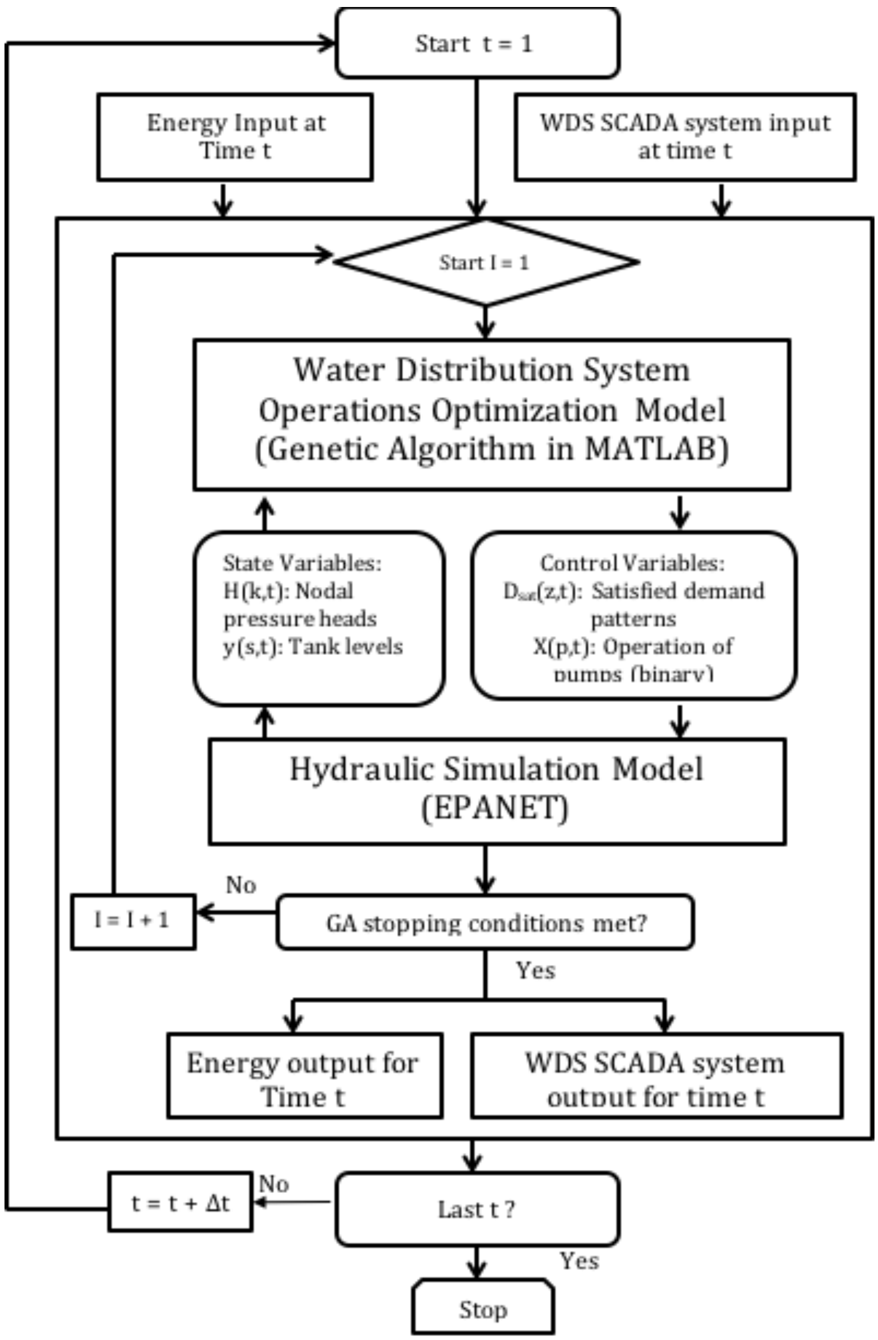


Figure 1-11 Flow Diagram for Genetic Algorithm Solutions

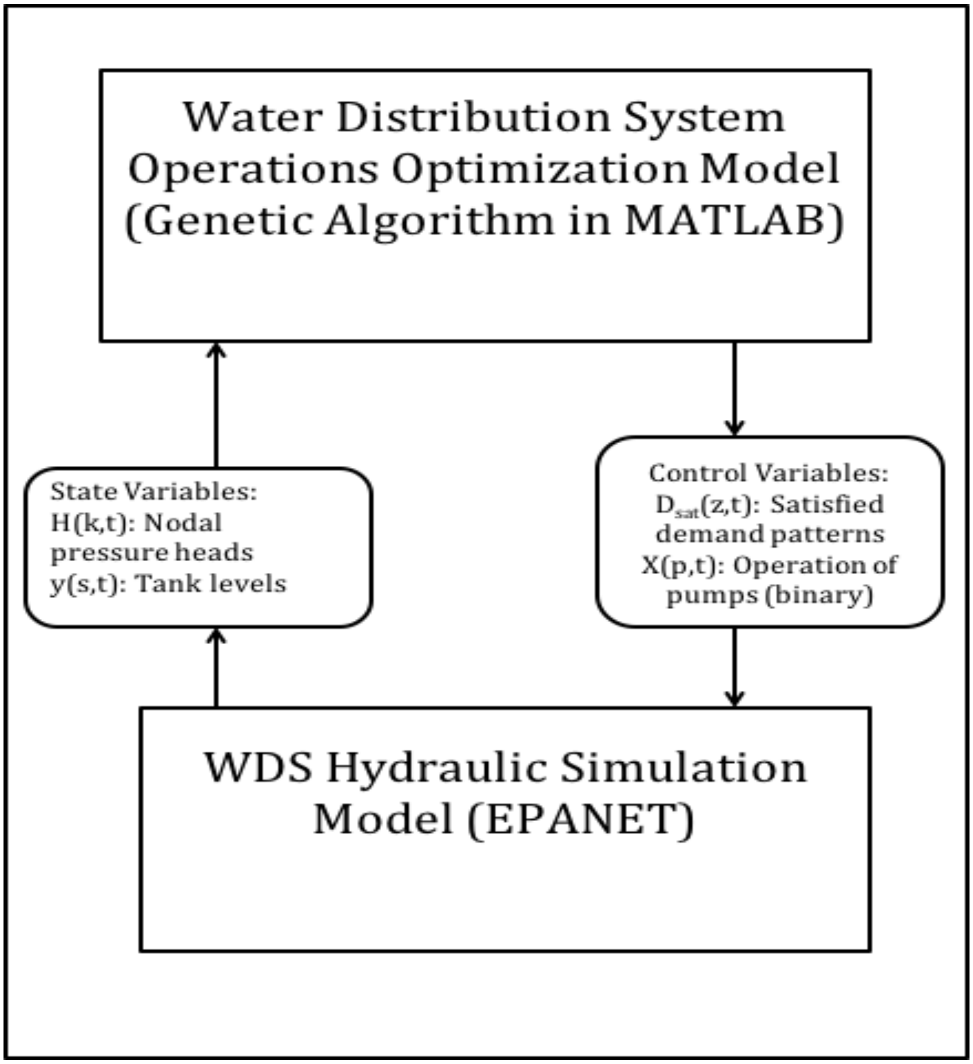


Figure 1-12 Schematic for Optimization – Simulation Interface

The model being considered here is the mixed integer non-linear problem (MINLP) for the operations and demand satisfaction for a water distribution system (WDS) being solved using genetic algorithm technique. The model consists of three weights: weight for demand satisfaction objective term, weight for pressure constraints penalty and weight for power requirement constraint. A sensitivity analysis was required for computing these weights to obtain a satisfactorily optimum solution.

1.6.7. Phase 7: Application of the Model for Realistic Example Systems

In this phase, the optimization-simulation methodology developed in phase 5 was applied for two realistic example systems. The first one involved a WDS without a power distribution system, while the second example system was a combined water - power system consisting of five power plants and two cities.

1.6.8. Phase 8: Development of a Combined Power – Water Nexus Simulation – Optimization Model

A combined model consisting of a middleware, a power optimization-simulation model and a set of water optimization – simulation models (developed in phase 5) was developed in this phase. This research phase included reviews and decisions regarding scenarios in which the various models dealing with real-time control and optimization of WDS with regards to contingencies arising out of the interdependencies between the power and water systems are to be used.

Given the fundamental understanding of the interdependency between the two infrastructure systems, a mathematical model which captures the interdependency and the associated operation of the two systems is being developed. Since it has been established that each thermoelectric generating station has at least two separate sources of water and storage for at least 15 days, it is apparent that the fast dynamics associated with power systems will not play a role in capturing the dynamics that describe the interdependency between the two systems. Hence, a series of static power flows with the appropriate commitment of generating units based on the time evolution of the operating horizon is used to represent the power system.

At each time interval, an optimal power flow is formulated in conjunction with some water delivery, pump demands, and the electric system load demand constraints is solved to obtain the schedule plant outputs. For simulation of the water distribution system, a reasonable practice is to make the various decisions with regards to pump operations on an hourly basis. There are numerous parameters such as pressures at nodes and flows in the links as well as the tank levels which are required to be computed. This warrants the use of a dynamic simulation over a long period for the WDS using EPANET. For every time interval, a quasi-dynamic hydraulic analysis of the WDS for an extended time-period is performed after receiving the input from the power optimization/simulation system. Results from the hydraulic analysis are interfaced with an optimization methodology (genetic algorithm) to determine the real-time optimal operation of the water distribution system under both normal conditions and emergency conditions with limited electrical input.

Phase 8(a): Computer coding for interfacing the power optimization – simulation model with water optimization – simulation methodology developed in phase 5 and 7

The simulation control by which the data management, file management, intermediate calculations and linking of the commercial software packages used was implemented with a software packaged developed using Python. The network coupling was reflected within this overarching framework by alternatively exchanging data between the power system and water network simulations. The optimization and network simulation for the power system was done using an appropriate solver within AMPL and with General Electric's PSLF (positive sequence load flow) software, respectively. The US EPA's EPANET software was used for the extended period simulation of the water distribution system being analyzed, while MATLAB was used to implement the optimization procedure and to interface the WDS simulation with the optimization procedure.

To demonstrate the operation of the two interdependent infrastructure systems, the methodology developed here was applied to several scenarios of a test system comprised of two combined networks. A representative, realistic water distribution system was used in conjunction with the IEEE 14 bus system to demonstrate the coupled network interactions. The first two scenarios considered here consisted of 'steady state' runs done for comparison, with one run reflecting the knowledge about conditions in the other network being taken into consideration and the other not taking this knowledge into consideration. Finally, scenarios which show the effects of a sustained power outage as well as another demonstrating a water shortage have been simulated.

Phase 8(b): Sensitivity analysis and extensive testing of model

The application of model formulated in phase 5 with genetic algorithm in MATLAB required an exhaustive sensitivity analysis for obtaining the most optimal results. The various parameters (settings) that are required for application of a genetic algorithm include population size (number of solutions considered in a iteration), number of generations (number of times selection of solutions is performed), number of stall generations required, fitness limit for iterations etc. The other parameters to be considered for a sensitivity analysis include the weights for the various penalty functions within the reduced objective. The computation of weights would be a required procedure for every WDS network being considered. Another important part of this phase was to perform an extensive testing of the model to study and verify the results of the sensitivity analysis and the overall model application.

1.6.9. Phase 9: Application of Combined Model for Normal as well as Contingency Scenarios of a Real System

This phase included application of the models developed as a part of phase 5 for realistic example water distribution systems using the computer codes developed in phase 8. The application aimed at studying results from the models in form of the pump schedules and demand satisfaction patterns to review the performance of these models for dealing with various contingencies and emergencies that can occur in operations of the water distribution system. Several contingency situations would be considered for this study including but not limited to failure of the electric grid to provide power in required amounts to the pumping stations, severe drought conditions leading to limited water availability, etc. The results were then studied to determine the targeted intervention in the operations of the WDS performed using the optimization models improved the resilience of the overall operations of the nexus.

Phase 9(a): Application of combined models for normal conditions of power and water availability in the water – energy nexus

The model was applied for two example networks under normal conditions of power and water availability. This application was performed to assess the applicability and performance of the methodology for optimization of the overall water – energy nexus operations.

Phase 9(b): Risk analysis and formulation of scenarios for water – energy nexus contingencies

For testing the applicability of the methodology developed in this research, it is necessary to predict and simulate the various foreseen contingencies that could plague the nexus in normal or emergency operations of the system. A risk-based approach is being used to formulate and predict these scenarios to achieve sufficient testing of the models for all foreseen conditions of the water – energy operations under emergency conditions. The scenarios were well defined to achieve a standardized set of emergency scenarios for which a certain combination of optimization – simulation models would be employed using the methodology developed herein. The scenarios include short term power outages due to component failures, long term power shortages of different magnitudes because natural disasters, short term limited water availability, long term limited water availability because droughts and uncertainty in the demand patterns. Several combinations of these scenarios could also be formulated for trials.

Phase 9(c): Application of combined models for various contingency scenarios

The model developed for combined optimization – simulation for the real-time operations of the water-energy nexus in phase 8 is being applied to the various scenarios formulated based on risk analysis in phase 9(b). The application was performed for two example WDS systems to assess and compare the results of this application with the state of art in practice.

1.6.10. Phase 10: Consideration of Water Age and Water Quality Constraint for the Model Developed in Phase 8

The main function of a WDS is to ensure a sufficient water supply at the consumer end both in terms of quality and quantity. Quality is affected by two things in a WDS, viz. the time which the water takes to travel through the distribution system (water age) and the leakages in the system. Contamination of water because leakages is taken care by the pressurized flow, which ensures flow out of the pipe rather than into the pipe. Therefore, a WDS has a lower pressure bound, to ensure public health safety from contamination of drinking water. The time spent by the water in the system could be simulated in two ways, viz. direct computation of water age or computation of concentrations of the disinfectant at different points in the system. In this phase of the research, both water age and concentrations will be considered as constraints to develop modified objectives for the GA.

1.6.11. Phase 11: Analysis for Resilience of Operations of the WDS.

Phase 11(a): Research on resilience of water distribution system operations optimized using the model developed in research phase 6(c) for pump controls and demand satisfaction

Phase 11(b): Research on resilience and vulnerability of water distribution system operations optimized using the combined model developed in research phase 8(a) for pump controls and demand satisfaction

1.6.12. Phase 12: Development of Contingency Plans, Operation Procedures and an Adaptive Management Strategy for the Real-time Operations of Water Distribution Systems under Limited Electrical Energy Input and Drought Conditions

The work done in the earlier phases achieves the second principle of earth systems engineering and management (ESEM) methodology, which includes evaluation of the technological fix. The landscape for this problem includes the overall water – energy nexus, while the actors/agents comprise of the various hardware and software components of both the independent systems including the respective distribution systems and supervisory control and data acquisition (SCADA) systems. The landscape change includes the various contingencies that can occur in any one of these systems, leading to a failure in the other due to the interdependencies. Several foreseen and unforeseen contingencies could occur in any of these systems as discussed earlier and therefore, an 'adaptive management' strategy is proposed to be formulated for this problem. Since the 'adaptive management' methodology is not a magic wand and comprehensive planning is required for implementing it, the proposed study would be aimed at looking for a broader plan/ methodology for applying the technological fix by formulating policies, contingency plans and operation procedures by considering the various ESEM principles for various foreseen and unforeseen extreme scenarios as a solution for the problem explained above. The proposed study would specifically include formulation of emergency response protocols including but not limited to use of the integrated mathematical models developed for the combined energy and water system. The study would only look at one facet of the overall nexus, i.e the water distribution system (WDS) operations and their effects on the overall water – energy nexus. The methodology proposed to be developed herein, if employed by entities like the Salt River Project (SRP) or Central Arizona Project (CAP), is expected to improve the overall sustainability of the water – energy nexus.

1.6.13. Phase 13: Consideration and Development of Simulation-Optimization Methodology for Pressurized and Open Canal Flow Components of the Overall Water Supply System from Source to the End User

A regional WSS may consist of several sources of water including groundwater wells, rivers, lakes etc. In general, when water is drawn from surface water sources, it is conveyed from the source to the water treatment plant as an open channel flow for cost considerations. As with pipes, the flow in canals could be controlled using sluice gates and controlling the lift pumps which feed a canal. The system considered in this research considers pressurized flow within the urban WDS i.e. the pressurized flow downstream of the water treatment plant (WTP) or the waste water treatment plant (WWTP), if reclaimed water is conveyed in the WDS. This phase of the research deals with extending the optimization – simulation methodology to include simulation and optimization of the open canal flow components of the WDS to facilitate a better control on the regional level WDS from the source to the consumer end.

2. STATE OF THE ART FOR WATER DISTRIBUTION SYSTEMS OPERATION

2.1. Water Availability and Drought Trends

The proposed research deals with a methodology for optimization and control of operations of water – energy nexus under extreme conditions of drought and limited power availability, hence it is necessary to establish a background for this research in form of the various predictions of future climate conditions in the western part of US, which is the study area for this research.

Cook et al. (2015) provides compelling evidence for prediction of extreme drought conditions in the coming years in the American Southwest and central plains. The study focuses on futuristic projection of climatic conditions in the American Southwest and the central plains using empirical drought reconstruction and three soil moisture metrics from 17 state of art general circulation models to show that these models project significantly drier conditions in the latter half of the 21st century as compared to the 20th century and earlier paleoclimatic intervals. The paper states that there are records of numerous droughts in the Western North America of extensive and persistent nature throughout the medieval era and these had a great impact on the societies and ecosystems therein. An attempt is made here to compare the 21st century drought projections with the paleo record, which is a challenge due to varying number of parameterizations and complexities of various land surface models of the GCMs. The study concludes that in the latter half of the 21st century, it is projected that the southwest would face both a reduction in the cold season precipitation and an increased evaporative demand in a warmer atmosphere. Over the central plains, it was observed that the precipitation responses across the models were inconsistent and the drying is primarily driven by increased evaporative demand. With the shift in the full hydroclimate distribution, the risk of decadal or multi-decadal drought occurrences increases substantially. It is thus demonstrated that the mean state of drought in the late 21^{st} century over the central plains and the southwest will likely exceed even the most severe mega drought periods of the medieval era.

Ault et al. (2016) give predictions of droughts for the American Southwest region with regards to the various climate change scenarios. The authors assert that a mega drought in the American Southwest would impose unprecedented stress on the limited water resources of the area, making it critical to evaluate future risks not only under different climate change mitigation scenarios but also for different aspects of regional hydroclimate. It is found in this paper that the changes in the mean hydroclimate state rather than its variability determines mega drought risk in the American Southwest. Regional temperature rise alone is found responsible for risk above 70, 90 and 99% by the end of the century, even if precipitation does not change considerably. The paper focused on characterizing mega drought risk as a function of variables that govern the balance of moisture at the land surface during climate change. The findings of this study by Ault et al. (2016) would have important implications for both mitigation and adaptation of the effects of climate change on the hydrologic cycle. The authors recommend a constellation of adaptation policies, including demand reduction and increased efficiency strategies, inter-basin water transfers, shifts to groundwater reliance, increased surface irrigation, and other management measures.

Cook et al. (2010) present a case for the various mega droughts predicted by IPCC (2007), while providing and discussing further evidence for the same predictions. The paper discusses about a multi-model assessment of projected changes in precipitation and surface water from 1999 to 2099 based on the medium A1B forcing scenario that increases greenhouse gas emissions until 2050 and gradually decreases them thereafter. The paper asserts that the most severe drought year occurred in 2002 based on intensity and spatial coverage, with more than 50% of the coterminous USA being under moderate to severe drought conditions. Cook et al. (2010) conclude that while there is no guarantee with regards to the response of the climate system to greenhouse gas, will result in mega droughts of the kind experienced in the past by North America, the IPCC predictions are not comforting and show a trend of the water availability going from bad to worst. Huber and Gulledge (2011) examine recent extreme weather events, their consequences, and links to larger statistical trends toward higher frequency and severity. The authors call for a probability-based risk management framework for adapting to and mitigating the effects of climate change. A probabilistic risk-based management framework for management of water distribution systems is presented and discussed in chapter 4 of this proposal.

Hoerling et al. (2012) investigate the reasons behind recurrence of droughts in the US Great Plains while branding the response of Great Plains climate to global warming as a key unresolved question. The study presents a parallel diagnosis of projected changes in drought as inferred from Palmer drought severity index (PDSI, Palmer, 1965), shown as an excellent proxy indicator for Great Plains soil moisture in the twentieth century. The authors explain that the applicability of PDSI breaks down in the twenty-first century, since it overstates surface water imbalances and implied agricultural stresses. The paper uses different drought indices to assess a possible explanation to inconsistency in trends for surface water balances among analyses.

2.2. Supervisory Control and Data Acquisition (SCADA) Systems

Supervisory control and data acquisition (SCADA) systems used in water distribution systems consist of several components such as remote terminal unites (RTUs), communications (telemetry transmission), a master station and a human – machine interface (American Water Works Association, 2010). Figure 2-1 gives a component level layout of a WDS SCADA system. The main role of the master station is to scan the RTUs, process the data, transmit operator commands and maintain a record of historical data. A centralized computer control is used in most SCADA systems, which controls several RTUs. In modern SCADA systems as depicted in Figure 2-1, a more distributed computer control is used because availability of much more compact and powerful computers. Currently, completely automated SCADA systems are not universally accepted, because of their susceptibility to attacks and concerns related to their reliability. But it could be reasonably argued that with the ever-increasing complicatedness of the WDS and an unabated demand for more optimal operations for a better sustainability of the system operations, implementation of a fully automated SCADA system is inevitable in future. In a SCADA system, the control can be remote or automatic with subsystems that consist of

- remote terminal units (RTUs),
- communications (telemetry transmission),

• a master station, and

• human–machine interface (through graphical format a central console).

The data acquisition in a WDS SCADA includes the various hydraulic and water quality parameters from sensor units at various locations within the WDS.

Modern water distribution systems have a high degree of complicatedness and hence require sophisticated mechanisms for their optimal control. An operator's primary responsibility is to supervise and control a WDS. American Water Works Association (2010, p. 209) define supervision as 'Supervision means examining systems performance information and deciding if it is acceptable while a setup for which a human operator evaluates the performance continually is known as an open-loop control'. The control instruments in a WDS enable the operators to change valve settings, pump settings (on and off) or otherwise adjust the system for efficient operation. For an acceptable level of performance, it is required that a water distribution system is operated properly. Most water utilities now use some form of a supervisory control and data acquisition (SCADA) system for control and operations of a WDS. Chase (2000) explains the various features of a state of art SCADA system used in US. The chapter explains the criteria for SCADA operations, emergency operations, monitoring of system performance using SCADA and gives a detailed technological anatomy of the various components involved in a WDS SCADA system.

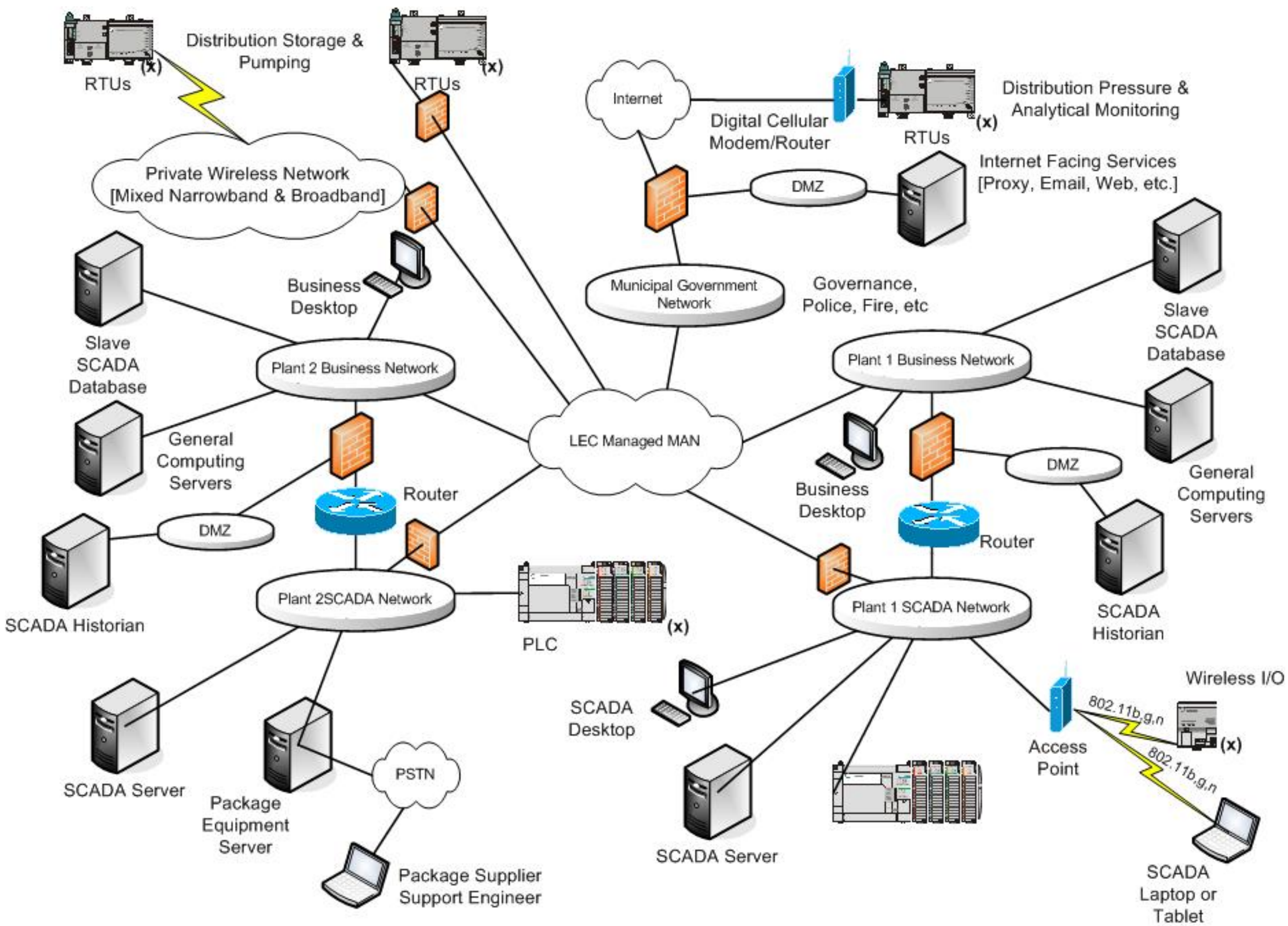


Figure 2-1 Components of a SCADA System (Source: CH2M Hill)

Barnes et al. (2004) review data related to Supervisory Control and Data Acquisition (SCADA) systems used to supervise and control domestic electric power generation, transmission and distribution has been presented in this study. Technical details for the types of systems, connections and a gap analysis of SCADA security loopholes is provided in this study. SCADA systems come in a myriad of types, sizes and applications. The report gives a list of SCADA system manufacturers and information about a multitude of SCADA systems. The electric power grid in US is made up of more than 3000 public private and government owned utilities and rural and municipal cooperatives. The report gives a detailed description of the power scenario in the US including generation, distribution and management of the power grid. Details regarding the SCADA hardware and operating systems available and how they are used for the management of the grid is also a part of the report. Information about the life of a SCADA system, electronic security features, defense tools and physical security features of the system are discussed in detail. The report also gives a gap analysis to study the various gaps in the efforts being dedicated to reducing and mitigating the risks of electronic attack to electric power systems.

Dobriceanu et al. (2008) gives a methodology employed to facilitate the use of SCADA systems for monitoring the water supply network, particularly to monitor the efficient pumping at the pumping stations in the water distribution systems. A DMS / SCADA type informatics system is proposed which would not only allow an optimized drive of the technological process but also facilitate a greater safety with regards to drinking water. In developing the model, various principles have been adopted by the author, including distributed processing open systems, principle of modularity, principle of autonomous and integrated working of equipment, principle of mutual settlement of equipment, principle of transparency in using and working, principle of best cost/ performance ratio. The aim of the model is outlined to provide effective monitoring, control and management for installations, management of installations besides the real-time operations, providing the required information for analyzing the behavior in operation and working out statistics related to the working of existing networks, providing the information for the superior dispatcher levels, a central level corresponding to the dispatcher. The author gives an architectural outline of the model as a system developed on equipment distributed network model based on the present standard level of computational technique. The monitoring and control system has the role to supervise the evolution of the technological process, to measure the consumptions and production and then to optimize the technological process for various functions given in the paper. Author also gives the various components of the system including transducers, Programmable Logic Controller (PLC), automat acquisition of specific parameters, local display etc.

Though SCADA systems are much more efficient in controlling and supervising a WDS as compared to their human counterparts, there are certain inherent threats in usage of such a system. One of the largest threats is the cyber-attacks on such a system. Most of the SCADA systems are computer network-based systems consisting of several nodes and a central server. Amin et al. (2013) discuss the threats related to cyber security of WDS and prevention techniques thereof. The reliability of a such a system largely depends on the cyber and physical security of the system. SCADA plays an important role in implementation of the various methodologies presented in this research.

2.3. Water Distribution Systems (WDS) Operations

Walski et al. (1987) explains “The battle of the network models”, which is the name given to a series of sessions held at the conference “Computers in Water Resources” at Buffalo, New York, in June 1985 to bring together researchers and practicing engineers for a critical appraisal of the current situation in pipe network optimization. In preparation of “The battle of the network models”, each of the participants solved a problem of sizing pumps, tanks and water mains for additions to the same hypothetical in-place water system. The systems given had features and problems typical of those found in real world. The problem focuses on the water distribution system of a hypothetical community, Anytown, USA. The town takes its water from a river and treats it at a central plant. Three identical pumps connected in parallel take water from the clear well at the treatment plant and pump it into the system. The town is said to be originally developed to the southeast of pipe link 28, with old cast iron pipes laid in this section having low Hazen-Williams C factors. Thereafter, the town began to grow in the northwest and west. Some industries have located near node 160 and a tank was also erected there. the utility has some troubles filling the tank. Nodes 65 and 165 re elevated tanks with 250,000-gallon capacity in each, while node 10 is a clearwell at the water treatment plant. The problem presented in the paper is to select new pipes, pumps and tanks. The pipes need to be cleaned and lined to meet minimum pressure requirements at minimum cost. Several important models concerning optimization efforts for WDS were presented in this conference proceedings.

Cullinane (1989) presented a paper aimed at developing a unified methodology for evaluation of urban water distribution system reliability. The methodology thus developed is useful for the planner or designer to design a distribution system within hydraulic constrains and incorporating the system and subsystem mechanical reliability. The methodology also provided for a system, which evaluates network and nodal reliabilities in the water distribution system. Finally, after considering various reliabilities the output of the system was selection of operation strategies for pumps and pump stations accommodating a reasonable number and variety of network complexities. This research provided for the first time a unified and accepted approach to evaluate the water distribution system reliability. Reliability was a major issue that the infrastructure in US was facing and the question was whether the urban infrastructures would provide adequate water supply for industrial development while maintaining high degree of reliability for provision of waters for drinking and public safety. Hence, it was necessary to develop a methodology which evaluates the reliability of the system. The author specifically states that no one definition will be appropriate for all systems and hence every system should be considered separately. The study conducted in this article involves evaluation of various factors responsible and affecting the reliability of the system. Mathematical formulation for probability and reliability computations has been applied in this study to evaluate the reliability or availability of the water distribution systems including various factors like age, repairs, modifications etc. A generalized reduced gradient (GRG) algorithm for reliability is also proposed in this work. A numerical representation of the hydraulic availability of the water distribution system has also been proposed in the dissertation. Successive quadratic programming techniques have been proposed for improving the execution time.

Rossman (2000) gives a detailed description of EPANET, which is a computer program that performs extended period simulation of hydraulic and water quality behavior within pressurized pipe networks. The manual for EPANET published by the USEPA, gives details about the development, capabilities, workspace, working etc. of the EPANET model. A tutorial is also a part of the model. The important capabilities of EPANET include hydraulic and quality modeling of a water distribution system. It also has other useful capabilities such as financial computations for pumping and pipe costs, etc. The document introduces the various capabilities of EPANET 2.0 version including the hydraulic and water quality features of the program. The important capabilities of EPANET include hydraulic and quality modeling of a water distribution system. It also has other useful capabilities such as financial computations for pumping and pipe costs, etc.

Awumah and Lansey (1994) presented a methodology of determining optimal pump operation schedules for municipal water-supply systems is presented in this study. in addition to minimizing the energy consumption costs, the model includes a constraint to limit the number of pumps switched on during the planning period. A two-level approach has been used which includes analysis of system hydraulics in an off-line mode to simplify the hydraulics to generate cost and hydraulic functions for an online model. There is a provision in the model to include various constraints such as tank levels, rate of change of tank levels, pump switches during each run. The model is applicable only to a limited number of pumps in the system. (Germanopoulos and Jowitt, 1992). give a LP model for determining a schedule of pumping on a 24-hour basis. The model considers various costs and factors associated with the electricity usage such as unit and maximum demand electricity charges, relative efficiencies of available pups, the structure of electricity tariff, the consumer-demand profile are taken into consideration. It is based on the hydraulic properties and operational constraints of the network along with some inherent assumptions such as a pumping schedule feasible in terms of reservoir storage also satisfies the nodal pressure amplitude constraints. Another important assumption in the model includes the flow and power consumptions are not affected by the pump and valve controls in the network except for the station under consideration. Though a reasonable validity for these assumptions is given in the paper, these assumptions affect the applicability of the model for various scenarios not discussed in the paper by Germanopoulos and Jowitt (1992).

Sakarya and Mays (2000) present a new methodology for optimizing the operations of water distribution system pumps considering the water quality requirements. The importance of consideration of water quality regulations in the pump operation optimization schedules is outlined in the paper. Hence, the methodology presented in the paper considers both hydraulic as well as the water quality requirements in preparing a pump schedule. The solution presented here is based on a NLP mathematical programming approach. The model considers three different objective functions to determine optimal pump operation for water quality purposes – 1. Minimize the deviations of the actual concentrations of a constituent, Minimize total operation time of pumps, 3. Minimization of energy costs. The paper proposes use of reduction technique as a solution algorithm instead of integrating the hydraulic simulation code to a non-linear optimization problem. Since, mathematical formulation of the pump operation problem results in a large-scale non-linear programming problem, the problem is reformulated in an optimal scheduling framework, which determines the optimal solution by integrating the water quality simulation code EPANET to an optimization code. Such a partitioning of the variables results in a large reduction in the number of constraints, because the hydraulic and water quality constraints are solved by the simulator leaving only the bound constraints to be solved by the optimizer. In this method, the state variable bounds might be violated making the optimal solution more difficult to determine. To overcome this problem, a penalty function method is used which uses state variable bound terms as penalty functions. Thus, the number of constraints is also reduced considerably.

Shamir and Salomon (2008) describe a method for near-optimal real time on-line operation of urban water distribution system. The method utilizes a reduced model (RM) to reproduce the performance over time with high fidelity with optimization by a genetic algorithm (GA). Optimal real-time operation of urban water distribution system aims at minimizing operational costs, while meeting the pressure requirements. Use of operational storage for shifting pumping to times of low energy costs leads to energy cost savings. The paper considers a 24-hours period as sufficient for the analysis and hence, it has been used throughout the study for demand forecast. The paper describes the functioning of the POWADIMA and a case study of its application for the Haifa system in Israel, consisting of 126 pipes, 112 nodes, nine storage tanks, one operated pressure reducing valve and 17 pumps in five pumping stations. A calculated savings of up to 20% as compared to manual operation, for the demand data of the year 2000 was achieved. This model replaces the artificial neural network (ANN) by a reduced skeleton model of the network that better meets the requirements for computational efficiency and other advantages. The validity of such a reduced model was ensured by a study of similarity of trajectory of tank levels over time in the reduced model with that calculated by the full model. The model was run for a 15-day simulation period with the full network model connected to the genetic algorithm. A saving of about 12% was achieved but the model required extensive computational time and hence the paper concludes that a full network model cannot be tolerated in the real-time operation. Thus, a reduced model giving reasonable results was concluded to be more efficient compared to the original full model.

Price and Ostfeld (2012) address a basic problem experienced in case of formulation of optimization model for a water distribution system. Flows are generally modeled using Hazen-Weisbach or Manning's equation, which are convex in nature. This creates a non-linearity in the optimization model and the pure linear programming techniques cannot be used in such a case. Generally heuristic methods like NLP-GRG or genetic algorithm etc. are used to solve such a problem. To make these optimization model solutions more accurate, it is necessary to formulate the non-linear components as linear equations.

Kessler et al. (1998) present a methodology for finding the optimal layout of a detection system in a municipal water network. The detection system considered consists of a set of monitoring stations aimed at detecting a random external input of water pollution. The paper presents a design methodology for detecting accidental contamination in municipal water networks. The methodology presented in this paper aims at identifying the best selection of monitoring stations, which allows capturing an accidental intrusion of contamination within a given level of service. The paper also gives an example model and a case study of Anytown, USA, which is a hypothetical model used to compare various optimization methodologies developed for a municipal water distribution system. This case study demonstrates the capability of the proposed methodology on a more realistic case.

Park and Liebman (1993) associate the reliability of water distribution systems with two types of failures including mechanical failure and hydraulic failure. The mechanical failure is associated with system failures due to pipe breakage, pump failure, power outages, control valve failure, etc. Hydraulic failures include system failures due to delivered flow and pressure head being inadequate at one or more demand points. These result from changes in demand and pressure head requirements. Hydraulic reliability is a measure of the performance of the water distribution system. The author associates the performance on two factors: interaction between the piping system, distribution storage, distribution pumping; system appurtenances such as pressure reducing valves, check valves etc. and reliability of the individual system components; spatial variation of demands in the system and temporal variation in demands on the system.

Bao and Mays (1990) define mechanical reliability as the ability of distribution system components to provide continuing and long-term operation without the need for frequent repairs, modifications, or replacement of components or subcomponents. Thus, it is the probability that a component or subcomponent performs its mission within specified limits for a given period in a specified manner. The objective of the study is to present a methodology to quantify the hydraulic reliability for a water distribution system considering failure and reliability. This involves developing a methodology based upon

Monte Carlo simulation, incorporate the uncertainties of future demand and pressure head requirements, investigate the impact of uncertainty and examine the sensitivity of reliability. The methodology developed by Bao and Mays (1990) can be used in the analysis of existing water distribution systems.

Lansey et al. (1989) state that the real issue of water distribution system reliability concerns the ability of the system to supply the demands at the nodes or demand points within the system at required minimum pressures. The conventional design process for water distribution systems is a trial and error procedure that attempts to find a design that represents a least cost solution that can satisfy demands. No attempt is made in such systems to analyze or define the reliability aspects of the designed system and have no guarantee that the resulting system is a minimum cost system. The real issue of water distribution system reliability concerns the ability of the system to supply the demands at the nodes or demand points within the system at required minimum pressures. The conventionally used trial and error method for design of water distribution systems does not guarantee that the resulting system is a minimum cost system. The paper presents a methodology, which incorporates the uncertainties in required demands, required pressure heads and roughness coefficients in the design of water distribution systems. The model presented in this paper is based on the premise that the water distribution systems are designed using specified demands, pressure heads, and roughness coefficients that are basically uncertain parameters that vary considerably with time.

Brion and Mays (1991) attempt to improve the pump operation efficiency focus on three different aspects: inefficient pump combinations, inefficient pump scheduling, and inefficient pumps. The non-linear programming optimization model to minimize pumping cost over a planning horizon subject to a constraint set that includes system constraints to account for the hydraulics involved in the water distribution system is mathematically a large model. Thus, the choice of constraints is dependent on the user's choice or system limitations. The optimal control problem for water distribution system is further complicated by the fact that the mathematical problem can be very large in the number of constraints, many of which are non-linear. This is complicated even more by the fact that many of these models are discrete. The author states that the numerous dynamic programming models proposed in the past suffer from the curse of dimensionality, limiting the size of problems (number of pumps, storage facilities, and size of network) that can be considered; as a result, the DP approaches are applicable only to very small problems. The methodology presented here overcomes the difficulties of these previous models.

2.4. Modeling of WDS – EPANET

With an unabated pace of urbanization that the humanity has experienced during the last few centuries, the water distribution systems became more and more complicated. This required development of techniques to study the functioning of the water distribution system for designing and operating them in such a way as to meet all the consumer demands as well as the pressure requirements in the system. Design of such systems is a tedious process and requires an intricate set of mathematical computations.

The performance of such systems varies with time and other conditions specific to the system and hence it is very difficult to test the system for safety against failure due to all such foreseen and unforeseen conditions. One method for this could be analyzing an existing similar system and making observations to avoid the existing errors, while other method frequently used is that of modeling.

Modeling is the process of mathematical or physical representation of a real-world system. Models based on analogy of water flows and electric current were used on a large scale in the later part of the 20th century. These models compared the flow of electricity in wires with the flow of water in pipes for simulation purposes, thus allowing a dynamic analysis of water flow. With development of better computing machines, mathematical models gained prominence over the physical models at the beginning of the 21st century. Robinson et al. (2012, p.3) explain the historic development of distribution system modeling. The authors explained the overall development of water distribution system modeling from the manual calculations in the pre-1970s era using the Hardy Cross method for single looped systems to simple software packages developed during the 80s and thereafter they explain the development of EPANET and related software packages for a more complete hydraulic analysis for complicated WDS.

Model-based simulation is a method for mathematically approximating the behavior of real water distribution systems (Walski et al., 2001). The various steps involved in preparing an optimization model given by Mays and Tung (2002) include:

1. Collection of data to describe systems
2. Problem definition and formulation
3. Model development
4. Model verification and evaluation
5. Model application and interpretation

Model development is an iterative process including mathematical description, parameter estimation, model input development and software development.

Robinson et al. (2012) presented a detailed description of the various computer models developed up to date for simulation of WDS and the various advances trends in WDS modeling. A typical computer model of WDS includes nodes and links among other components. Pipes and pumps are generally represented as links, while demand and supply points are represented as nodes. Various characteristics are required to be defined for the numerous components of the water distribution system. These include dimensions for pipes, tanks, reservoirs, etc.; demands for various nodes; system curves for pumps; minimum and maximum levels for tanks; elevations and other topographic features for the network.

What is EPANET?

EPANET is a computer program, which performs steady – state or extended period simulation of hydraulic and water quality behavior within pressurized pipe networks (Rossman, 1994). It is a worldwide-accepted software and is used on a large scale by numerous designers as well as government agencies around the globe. EPANET provides an integrated environment for editing network input data, running hydraulic and water quality simulations and viewing the results in a variety of formats.

EPANET models a water distribution system as a collection of links connected to nodes. The links represent pipes, pumps, and control valves. The nodes represent junctions, tanks, and reservoirs. Figure 2-2 illustrates how these objects can be connected to one another to form a network.

Hydraulic Modeling Capabilities (Rossman, 2000)

Full-featured and accurate hydraulic modeling is a prerequisite for doing effective water quality modeling. EPANET contains a state-of-the-art hydraulic analysis engine that includes the following capabilities:

- Places no limit on the size of the network that can be analyzed
- Computes friction head loss using the Hazen–Williams, Darcy–Weisbach, or Chezy– Manning formulas
- Includes minor head losses for bends, fittings, etc.
- Models constant or variable speed pumps
- Computes pumping energy and cost
- Models various types of valves including shutoff, check, pressure regulating, and flow control valves
- Allows storage tanks to have any shape (i.e., diameter can vary with height)
- Considers multiple demand categories at nodes, each with its own pattern of time variation
- Models pressure-dependent flow issuing from emitters (sprinkler heads)
- Can base system operation on both simple tank level or timer controls and on complex rule-based controls

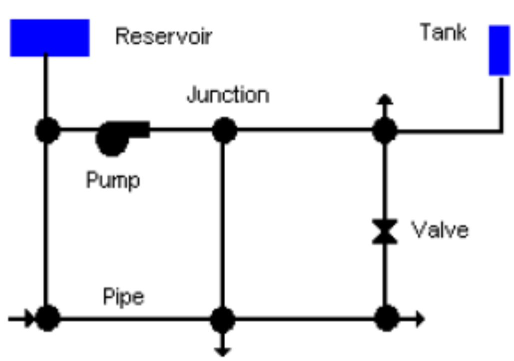


Figure 2-2 Physical Components of a Water Distribution System (Rossman, 2000)

EPANET uses a hybrid node-loop approach to solve the conservation of flow and energy equations, which describe a pipe-network at any given point. Rossman (2000) explains the solution method used in EPANET for the set of non-linear pipe-network equations. The distribution of flow throughout the network must satisfy the conservation of mass and the conservation of energy which are defined as the hydraulic constraints. The conservation of mass at each junction node, assuming water is an incompressible fluid, is

$$\textstyle\sum_i (Q_{i,k})_t - \sum_j (Q_{k,j})_t - Q_{k,t} = 0 \qquad \forall\ k = 1, ..., K \text{ and } t = 1,.., T \qquad (2\text{-}1)$$

where $(Q_{i,k})_t$ is the flow in the pipe m connecting nodes i and j at time t (gpm) and Q_{kt} is the flow consumed (or supplied) at node k at time t (gpm); K is the total number of nodes in the system.

The conservation of energy for each pipe m connecting nodes i and j, in the set of all pipes, M is,

$$H_{i,t} - H_{j,t} = f(Q_{i,j})_t \qquad \forall\ i, j \in K \text{ and } t = 1, ..., T \qquad (2\text{-}2)$$

Where, $H_{i,t}$ and $H_{j,t}$ are the pressure heads at nodes i and j respectively.

The total number of hydraulic constraints is (K+M) T, and the total number of unknowns is also (K+M) T, which are the discharges in M pipes and the hydraulic grade line elevations at K nodes. The pump operation problem is an extended period simulation problem. The height of water stored at a storage node at time-period t, y_{st}, is a function of the height of water stored from the previous time-period which can be expressed as,

$$y_{s,t} = f(y_{s,t-1}) \qquad \forall\ s = 1, ..., S \text{ and } t = 1, ..., T \qquad (2\text{-}3)$$

The bounds on the level of water storage in a tank s for time t is

$$\underline{y}_{s,t} \le y_{s,t} \le \overline{y}_{s,t} \qquad \forall s = 1, ..., S \text{ and } t = 1, ..., T \qquad (2\text{-}4)$$

where $\underline{y}_{s,t}$ and $\overline{y}_{s,t}$ are the lower and upper bounds, respectively of the elevation of water stored in node s at time t, y_{st}. These limits are normally due to physical limitations of the storage tank.

For performing a hydraulic analysis for a piped water distribution system, EPANET implicitly solves equations 2.1 – 2.4 for all the nodes and links in the system for every time during the simulation run.

In any municipal water distribution system, large variations are experienced as far as water demands are considered. Large seasonal and hourly variations in the trends of domestic water use are experienced in most of the urban areas across the world. These variations follow a daily as well as seasonal trend. Peak seasonal demands are observed during summers (June through September) in most of the places, while a diurnal pattern of water use could be observed in a highly urbanized community. The variations of demands during the daily operations are represented in the form of a demand pattern. Demand patterns are generally expressed in terms of multipliers, which are defined as the ratio of the hourly demand and mean demand for a node.

Along with the variations in the demand patterns for daily operations in a water supply system, another important consideration is the variations in the capacity of the water distribution system to meet these demands effectively during an operation time-period. This is dependent on several factors including the storage capacity (tanks) available, pumping capacity and schedules, water availability, power availability etc. With the decreasing trends of water availability and an ever-increasing trend of population and demands, large deficits could be observed in many water systems across the globe as far as the demands and the demand satisfaction capacity are considered. This trend is more ominous in case of areas characterized by a natural scarcity of water. For effective and optimal functioning of a water distribution system, it is important that efforts are made to operate the water distribution system minimizing the deficit in the distribution system while considering the various hydraulic and quality requirements for the system.

Water Quality Modeling Capabilities of EPANET (Rossman, 2000)

In addition to hydraulic modeling, EPANET provides the following water quality modeling capabilities:

- Models the movement of a nonreactive tracer material through the network over time.
- Models the movement and fate of a reactive material as it grows (e.g., a disinfection by-product) or decays (e.g., chlorine residual) with time
- Models the age of water throughout a network
- Tracks the percent of flow from a given node reaching all other nodes over time
- Models reactions both in the bulk flow and at the pipe wall
- Uses n-th order kinetics to model reactions in the bulk flow
- Uses zero or first-order kinetics to model reactions at the pipe wall
- Accounts for mass transfer limitations when modeling pipe wall reactions
- Allows growth or decay reactions to proceed up to a limiting concentration
- Employs global reaction rate coefficients that can be modified on a pipe-by-pipe basis
- Allows wall reaction rate coefficients to be correlated to pipe roughness

- Allows for time-varying concentration or mass inputs at any location in the network
- Models storage tanks as being either complete mix, plug flow, or two-compartment reactors

2.5. Use of Evolutionary Algorithms for Optimization of WDS Design and Operations

An evolutionary algorithm uses mechanisms inspired by nature and solves problems through process that emulate the behavior of living organisms. Evolutionary algorithms are inspired by Darvinian theory of evolution and perform heuristic computations to reach an optimal solution. Evolutionary algorithms are extensively used for optimizations of water distribution designs and operations. Barán et al. (2005) present a methodology for use of multi-objective evolutionary algorithms (MOEAs) used to solve an optimal pump-scheduling problem with four objectives of minimized electric energy cost, maintenance cost, maximum power peak and level variation in a reservoir. Water distribution systems are key elements of water infrastructure and require a significant investment. As water demand grows, these systems become larger and more complex. Optimizing the pump scheduling has proven to be a practical and highly effective method to reduce the operational costs without making changes to the actual infrastructure of the whole system. This optimization process may become highly complex in large distribution systems. A pumping station consists of a set of pumps with different capacities and they operate on a schedule to pump water to one or more reservoirs. This paper presents an analysis of an optimal pump-scheduling problem as a multi-objective optimization and its solution using MOEAs, since due to great advances recently achieved in the field of evolutionary multi-objective optimization, their undoubted usefulness and the complexity of the pump scheduling problem. The author states that great advances in the field of evolutionary multi-objective optimization, these are undoubtedly useful in providing solutions for the relatively complex pump scheduling problem. Six different algorithms were implemented and combined with a heuristic method that handles problem constraints. MOEAs unlike the traditional methods of optimization, optimize the different objectives simultaneously without aggregation. Four objectives are considered for the study including electric energy cost, pump's maintenance cost, peak power and level variation in the reservoir.

Ostfeld and Tubaltzev (2008) present an ant colony optimization algorithm for design and operation of pumps for a WDS. They explain ant colony optimization technique as a relatively new meta-heuristic stochastic combinatorial computational discipline inspired by the behavior of ant colonies; ant deposit a certain amount of pheromone while moving, with each ant probabilistically following some direction rich in pheromone. This behavior has been used to explain how ants can find the shortest path between their nest and a food source, and inspired development of ant colony optimization. The optimization problem presented in this paper includes linking an ant colony scheme with EPANET for minimization of the systems design and operation costs. The decision variables for the design are the pipe diameters, the pumping stations maximum power and the tanks storage, while the operation these include the pressure heads and water levels in the tanks at all loadings. The least-cost design problem is to find the water distribution system component characteristics: pipe diameters, pump heads and maximum power, and tank storage which minimize the total system cost, such that constraints at the consumer nodes are fulfilled and hydraulic laws are maintained. The author gives a classification for various models developed till date for optimization of water distribution systems. These include decomposition systems using LP methodology, linking simulation with non-linear programming methods, non-linear programming methods and methods employing evolutionary algorithms. Genetic algorithms are domain heuristic independent global search techniques that imitate the mechanics of natural selection and natural genetics of Darwin's evolution principle. This paper describes the development and application of an ant colony based algorithm for the conjunctive least cost design and operation of water distribution systems expanding the application of ant colony optimization to multiple extended periodic loading conditions and to water distribution systems with pumping stations and elevated storage. The objective of the model is to minimize the total cost of designing and operating the system, while delivering the consumers required quantities and acceptable pressures.

Behandish and Wu (2014) state that drinking water and wastewater utilities account for about 3% to 4% of the total energy use in the United States and produce about 45 million tons of greenhouse gas emission annually. The problem of operation optimization is most frequently addressed by pump scheduling, which involves either implicit control rules or explicit time-based specifications on when to turn pumps on and off optimally. Pump scheduling with direct application of hydraulic solvers is computationally intensive when applied to models of large utilities. To overcome this difficulty, parallel computing technology has been utilized. In addition to pump schedule, other hydraulic parameters including operation ranges of storage tank levels could also be used as decision variable in optimization of the energy requirements. Some tanks might have more impacts on energy savings than others if they are filled when electricity is inexpensive and drained during peak-demand periods. This if optimally designed could lead to lower energy consumption. The paper focuses on application of the generalized multi-artificial neural network (ANN) meta-modeling technique developed by the authors in combination with a modified Generic Algorithm (GA) to solve for this set of decision variables including pump operational costs.

The operation of water distribution systems impacts the water quality in these systems. There have been few attempts to optimize the water systems operations for both hydraulic and water quality performance Goldman and Mays (1999) state that such studies conducted in the past were limited to simplified systems. A new methodology is presented by Goldman and Mays (1999) that formulates the water distribution system problem as a discrete time optimal control problem linking the method of simulated annealing with EPANET for optimal operation of water distribution systems for both water quality and hydraulic performance. Simulated annealing allows optimization of a variety of objective functions and can consider many modifications to operational conditions without reprogramming of the optimization procedure. The methodology presented here was applied to two water systems as examples including the northwest pressure zone in Austin, Texas and the North Marin Water District, Novato, California.

2.6. Genetic Algorithms (GA) for WDS operations

The genetic algorithm (GA) is a methodology for solving both constrained and unconstrained optimization problems that is based on natural selection, the process that drives the biological evolution. The genetic algorithm repeatedly modifies a population of individual solution and at each step produces better solutions known as offspring. Vamvakeridou-Lyroudia et al. (2007) give an approach to optimize water distribution system design using floating-on-the-system tanks as decision variables aiming to bridge the gap between traditional engineering practice and mathematical considerations needed for genetic algorithms (GAs). The purposes of provision of floating-on-the-system tanks are energy head (pressure) regulation through their water level and storage capacity though their volume. Large tanks, with considerable volume, can perform both functions. Small tanks are generally used solely for head regulating purposes. Though pipes are the major cost affecting components of a distribution system, tanks affect the overall design and hence their placement and design is significant as far as cost and quality performances are considered. The author gives an account of the complexities associated with mathematical simulation using tanks as components and decision variables for a water distribution system optimization model. These include peak loadings, operational aspects, extended period operations etc. Also, tanks should be filled at off peak periods using pumps. This adds pumps as decision variables as well. Genetic algorithms (GA), are widely used for water network design optimization and using these the author could include tanks and storage as decision variables in the model. It is also stated that if tanks are to be simulated using GA process, a detailed extended period simulation with a small-time step should be performed.

Van Zyl et al. (2004) explain the development of an optimization model for operation of water distribution systems using a hybrid methodology including the GA technique and the hillclimber technique of optimization. In most of the optimization models, it is required that the problem be simplified through assumptions, discretization or heuristic rules but genetic algorithms (GAs) do not require such simplification measures, giving them a significant advantage over the other optimization techniques in finding a near optimal solution for most of the problems. The paper outlines certain difficulties encountered in optimization of water distribution systems, including variable demands and electricity tariffs over a typical operation cycle, minimum level of water to be maintained, pressure in the system and the number of pump switches. The drawback with a simple genetic algorithm system is that it requires a high number of function evaluations to achieve convergence. In this study, the efficiency of GA operational optimization of water distribution system was improved by developing a hybrid optimization method, which combines Gas with a hillclimber search strategy. Thus, GA can identify the region of optimal solution efficiently, but is much less efficient in finding the optimal point inside the region. A hillclimber method explores in the vicinity of a solution for improvements using a specific search strategy, to seek a minimum or maximum in the optimization process. hillclimber technique is efficient in finding the local optimum but is not able to escape the attraction basin of the local optimum to explore the other regions of the solution space. hillclimber techniques are therefore strong where the GAs are weak. This study uses a hybrid method to utilize the advantages of both these techniques. Operational optimization is used to provide an acceptable level of service to the customer within system constraints and legal regulations. It is required that a balance be struck between the cost and risk. Cost will play a dominant role in most operational optimization problems. The most common potential reductions in the operational costs in water distribution system are by scheduling the pumps to reduce electrical energy costs. In addition to this, there are other ways to reduce the costs including use of the cheapest water source, water loss reduction, minimization of number of pump switches, etc. Tank level controls trigger control actions in the distribution system when tank water levels reach certain predetermined values and are widely used in practice, due to their simplicity and proven robustness. Tank level controls are normally used in pairs, with one control triggering an action (such as switching pump on) and the other triggering an opposite option (switching pump off). The operational cost or total pump energy cost, of each set of variables was calculated by doing an extended period simulation of the system. The study uses two operational constraints for the optimization model. The first one being that of the tank water levels to balance over the run and the second one was to limit the number of pump switches in a 24-hour run to avoid an increase in maintenance costs incurred because wear and tear caused by excessive switches.

Boulos et al. (2001) investigate the integration of on-line telemetry and optimal computer control systems to reduce operating costs and provide more reliable operations have started on a large scale. A water utility spends about 65% of its annual budget on energy costs associated with the operations (Boulos et al., 2001). This paper discusses a new management model H_2ONET scheduler for optimal control and operation of water distribution systems. The H_2ONET scheduler casts the optimal control problem as an implicit non-linear optimization problem subject to both implicit and explicit constraints. It computes the optimal pump schedule for each pump or a group of pumps based on specific control time span, such that the overall energy cost is minimized. Pumps are normally grouped together based on their known characteristics such as location, pumping capacity and common control components (storage tanks). The objective of the optimal control problem is to minimize the energy costs while satisfying the hydraulic requirements of the system.

Abkenar at al. (2015) present a methodology for evaluation of genetic algorithms prepared for pump control optimization. To find the optimal solutions (in terms of sustainability) optimization routines should minimize energy demand, cost and pollutant emissions. The author states that finding the optimum solution for this type of non-linear problem with multiple constraints using traditional deterministic methods is challenging and has been a focus of extensive research and that all the efforts related to use of traditional methods to solve the NLP problem are limited to small systems, thus reducing the sample space. Among the evolutionary methods, GA is the most extensively used method for pump schedule optimization. GA searches for the global optimum solution over the whole solution space, instead of focusing on a part or boundaries of solution space. This leads to a local optimum, rather than a global optimal solution. In the GA, the study presented by the author, a random group of solutions is selected as the initial (trial) population. Each solution is a pumping schedule. During the parental selection process, a group of best solutions will be selected to form each subsequent generation. By repeating this process over multiple generations, the GA moves towards an optimal solution. The number of solutions in a generation (population size) is one of the important parameters that influence optimization by GA. To prevent searching from all possible solutions, evolutionary algorithms such as GA explore only a portion of the space and migrate towards an optimal solution.

3. MODEL FOR OPTIMAL OPERATION OF WATER DISTRIBUTION PUMPS WITH UNCERTAIN DEMAND PATTERNS

3.1. Summary

An optimization model for pump operation based upon minimizing operation and maintenance costs of pumps for a specified demand (load) curve is presented. The purpose of this model is to determine pump operation to meet the known consumer demands as well as to satisfy the pressure requirements in the water distribution system. In addition, constraints on the number of pump ('on-off') switches are included as a surrogate to minimizing the maintenance costs. This model is a mixed integer nonlinear programming (MINLP) problem to consider the uncertainty in demand using a chance constraint formulation of the demand constraint. The optimization model was solved using the LocalSolver option in A Mathematical Programming Language (AMPL). The model was first applied to the operation of the example pumping system for an urban water distribution system (WDS) illustrating a reduction in operation costs using the optimization model. The optimization model with the chance-constraint on meeting demand was applied for a range of demand satisfaction uncertainties. A decrease in the operation costs was observed with an increased uncertainty in demand satisfaction, which shows that the model further optimizes the operations considering the relaxed constraints. Model application could be extended to operations of pumping systems during emergencies and contingencies such as droughts, component failures etc.

3.2. Background

Operation of pumps is a major cost consideration for water distribution systems. Pump operations are optimized to minimize power usage, particularly, during peak hours of operation. Goldstein and Smith (2002) predicted the future electricity requirements for the first half of the 21st century to be around 4% of the total energy produced in the United States, while the average energy requirement for cooling water required for all types of power plants is estimated at around 0.5% of production.

A typical pumping plant may include several pumps operating in parallel to meet the demands in the network. A pumping plant should be designed to operate to meet the maximum quantities of forecasted normal and emergency demands in the system with a certain factor of safety. Although a system is designed for a larger discharge, the demands observed on a day-to-day basis are much smaller than the maximum design discharges. In addition to the excess pumping capacity provided in the system as a factor of safety, a typical water distribution system also consists of elevated storage. It is a general practice to pump water to these storage systems during the non-peak hours of power consumption for economic reasons. This storage water is then used for supply during the peak hours making the operations of the pumps in the system time dependent. The state of the art in practice is to prepare pump operation schedules (switch on and off times) based on the levels of water in the storage tanks. Whenever the water level in the tank falls below a certain amount, the pumps are turned on to fill the tank. This could lead to rapid switching on and off of the pumps causing excessive wear. Excessive wear then implicitly results in higher maintenance costs.

3.3. Previous Optimization Models for Pump Operation

A few previous efforts for optimizing the controls for pumps in a water distribution system (WDS) are explained here. Zessler and Shamir (1989) used progressive optimality, an iterative dynamic programming method for optimal operations of a water distribution system (WDS). Brion and Mays (1991) give a methodology based on solving a large-scale nonlinear programming problem while Germanopoulos and Jowitt (1992) used a simple linear programming approach for optimal pump scheduling for a water distribution system. Ormsbee and Reddy (1995) developed a nonlinear heuristic is developed for use in obtaining least-cost pump-operations policies for multisource, multi tank water-distribution systems. Sakarya et al. (1998, 1999) give a methodology using a non-linear programming approach for determining the optimal operation of water distribution systems for water quality purposes. Goldman and Mays (2005) explain and exemplify the use of simulated annealing technique for optimizing the operations of water distribution system considering water quality requirements.

Van Zyl et al. (2004) explain the application of a hybrid genetic algorithm for operational optimization of water distribution systems. Barán, et al. (2005) used multi-objective evolutionary algorithms for solving an optimal pump-scheduling problem with four objectives for minimizing electric energy cost, maintenance cost, maximum power peak and level variation in reservoirs. Goldman and Mays (2005) used a simulated annealing approach for operations of pumps considering the water quality requirements for WDS. Ostfeld and Tubaltzev (2008) present a model based on ant colony optimization technique for least cost design of pumps in a WDS. Costa et al. (2010) present a genetic algorithm connected with a hydraulic simulation model (EPANET) for optimizing the energy costs for pumping. Kurek and Ostfield (2013) present a multi-objective methodology for pump operations optimization and tank sizing considering the water quality in a WDS. Ghaddar et al. (2014) used a Lagrangian decomposition coupled with a simulation model for optimization of pump operations. Boulos et al. (2014) presents a case study for modeling of real-time control of WDS pumps for city of Las Vegas. Fayzul et al. (2014) present methodologies for obtaining a warm or initial solution for use of evolutionary algorithms for optimal real-time operations of a WDS.

Jamieson et al. (2007) present a conceptual methodology for determining feasibility and efficacy of near – optimal real-time operations for WDS. Another model for near-optimal controls for WDS using a genetic algorithm (GA) approach is presented by Shamir and Salomons (2008). Kang, et al. (2014) present an optimization model for real-time controls of WDS minimizing the operation costs. Optimization models such as Ozger and Mays (2005) provide a methodology for the location of isolation valves for security purposes using a simulated annealing approach interfaced with EPANET.

Boulos et al. (2001) investigated the integration of online telemetry and optimal computer control systems to reduce the operating costs and provide reliability in operations. The model minimizes the energy costs while satisfying the hydraulic requirements of the system. Little effort has been reported to develop a model for operation of pumping stations considering a variation or uncertainty in the demand satisfaction for a water distribution system. A novel approach of using a chance constrained mixed integer non- linear programing (MINLP) methodology for optimizing the operations of a pumping system is presented herein.

3.4. Optimization Model Development

3.4.1. Pump Operation Costs

Power costs depend primarily on the pricing policy of power utilities. The costs are not constant throughout the day and may vary hourly depending on the expected consumption levels during that hour. Generally, it is observed that the power costs follow a diurnal curve similar to a typical water consumption curve (Richardson et al., 2010). The best management practice for pumping in such a case would be to avoid or minimize pumping during the peak times of the day. While reducing the total operation (power consumption) cost, the model also attempts to limit the total number of switches, where a switch is defined as a pump being turned on. Limiting the number of switches implicitly results in lower maintenance costs.

Awumah and Lansey (1994) presented a methodology to optimize the operation of pumps for maintenance costs adopting a two-level approach whereby the system hydraulics are analyzed in an off-line mode to generate simplified hydraulic and cost functions for an on-line model. An operation schedule in which the pumps are switched on and off several times might reduce the overall energy costs, but excessive pump switches are known to result in high costs related to the wear and tear of the pumps. Maintenance cost is not directly evident as it could vary considerably with a change in the operating conditions of the pumps. The cost of unscheduled pump maintenance is often the most significant maintenance cost of utilities and the failure of mechanical seals and bearings are also among the major costs. The frequency of unscheduled maintenance is directly proportional to the number of switches in a pump's lifespan. Herein pump switches (Sw_p) are a surrogate measure of the intangible wear and tear caused during the operation of the pumps in a water distribution system. A switch (Sw_p) is defined as the number of times a pump 'p' is switched on during the total simulation time period.

3.4.2. Objective

The objective function to minimize pump operation costs is based upon minimizing the energy costs for operation is expressed as

$$\text{Minimize } Z = \sum_{p}^{p=P} \sum_{t}^{t=T} \frac{X_{p,t} Q_{p,t} H_{p,t} P_t \, \Delta t}{3956 \eta_p} \tag{3-1}$$

where Z is the total cost incurred for operations of all the pumps, $Q_{p,t}$ is the pump p discharge during time period t, $H_{p,t}$ is the pump head at time t, Δt is the unit time step for hydraulic computations, η_p is the mean pump p efficiency derived from discharge – efficiency curves, $X_{p,t}$ is a binary (0 or 1) variable depicting whether a pump p is switched on (1) or off (0) during a particular time t, and P_t is the unit cost of power per during time t.

3.4.3. Demand Satisfaction

The basic requirement of the water distribution system is that all the nodal demands are satisfied all the time, expressed as

$$Q_{prod} Pat_t \leq \sum_{p}^{P} \left(Q_{p,t} X_{p,t} \right) \tag{3-2}$$

where Q_{prod} is the average production required and Pat_t is the production or load pattern coefficient for the particular time t.

Head-discharge relationship adopted herein for pumps is

$$H_{p,t} = c_p Q_{p,t} + a_p \quad (3\text{-}3)$$

where c_p and a_p are the coefficients for a particular pump p. The head bounds constraint for the p-th pump at time t is as follows:

$$h_{min} \leq H_{p,t} \leq h_{max} \quad (3\text{-}4)$$

where h_{min} and h_{max} are respectively the minimum and maximum limits on allowable heads.

3.4.4. Switch Constraint

For the purposes of this study, the variable Sw_p is an integer variable, defined as the number of times a pump is switched on during the total simulation time period.

$$Sw_p = \sum_{t}^{t=T} \max(0, \; X_{p,t} - X_{p,t-1}) \quad (3\text{-}5)$$

which is a conditional (if – then) type of constraints.

where Sw_p is the total number of times a pump is switched on during the total simulation time period.

3.4.5. Switch Limit Constraint

The number of pump switches can be used as a surrogate measure for the maintenance cost of the pumps. The following constraint can be used to limit the number of times a particular pump is switched on during a pump schedule period.

$$Sw_p \leq Swmax_p \tag{3-6}$$

where $Swmax_p$ is the maximum number of daily pump switches allowed for pump p.

The above optimization model (equations 3.1 through 3.6) is a mixed integer nonlinear programming problem with equation 3.5 used to compute the number of pump switches Sw_p for pump p. The decision variables are the continuous variables $Q_{p,t}$, $H_{p,t}$ and the integer variables $X_{p,t}$ and Sw_p. This model was solved using the localsolver (simulated annealing) in AMPL.The solution methodology should be capable of solving a conditional type of constrain

3.4.6. Expression of Model for Uncertainty in Demand

The real issue of water distribution system reliability concerns the ability of the system to supply the demands at the nodes or demand points within the system at required minimum pressures (Mays et al. ,1989). An attempt has been made to address this uncertainty by developing a stochastic optimization methodology. A variation of several percent is generally observed in the demand pattern for a water distribution system. In emergency situations and during shortage of water, a deficit in the demand satisfaction is generally acceptable.

The chance constrained optimization model is based upon the assumption that the variations of demand satisfaction are normally distributed. For a normally distributed variation in demand. The chance constraint is formulated so that the demand is considered as a random variable with mean μ and standard deviation σ as it cannot be assessed with certainty. Equation 3.2 can be written in probabilistic form as:

$$\text{Probability}\left\{\sum_{p}^{P}\left(Q_{p,t}X_{p,t}\right) \geq Q_{prod}Pat_{t}\right\} \geq \alpha \tag{3-7}$$

where α is the specified reliability at the t-th time. The deterministic equivalent of equation 3.7 is the chance constraint used in the optimization model. (see Mays and Tung, 1992)

$$\mu + z(\alpha)\sigma \leq \sum_{p}^{p=P}(Q_{p,t}X_{p,t}) \tag{3-8}$$

where μ is the mean of the production demands, σ is the standard deviation of production demand and z(α) is the cumulative distribution function or the quantal function for normal distribution interpolated from standard normal distribution tables for specific values of α.

Optimization model

The modified objective function for this model is expressed as

$$\text{Minimize Cost} = \sum_{p}^{p=P}\sum_{t}^{t=T}\frac{X_{p,t}Q_{p,t}H_{p,t}P_{t}\Delta t}{3956\eta_{p}} \tag{3-9}$$

The constraints are equations 3.2 through 3.6 and 3.8 with continuous decision variables $Q_{p,t}$, $H_{p,t}$ and integer variables $X_{p,t}$ and Sw_p.

3.5. Model Solution Method

Two models are presented here, one for the case of certain demands (equation 3.2 through 3.6), while the other is for an uncertainty of the demands in the system. Both of these models are mixed-integer nonlinear programming (MINLP) problems. AMPL is a high level mathematical programming language used for solving large scale optimization problems including problems requiring linear, nonlinear or heuristic solution methodologies (Fourer et al. ,1993). The optimization models described above were solved using the AMPL system (www.ampl.com) (Gay, 1997). Benoist et al. (2011) explain the various capabilities of this solver for a MINLP problem consisting of binary variables and conditional constraints. Benoist et al. (2010) give the details of development and techniques used for LocalSolver 1.x solvers. This solver uses a heuristic (simulated annealing in combination with other techniques) to find a local optimum and an artificial neural network (ANN) technique to find the global optimum solution for a problem. The Local Solver is a solver in AMPL is capable of solving MINLP optimization problems having if-then-else constraints.

The data required for the model includes the characteristics of the pumps as well as the network. The inputs to the model include average production for the service area (Q_{prod}), characteristic pump curves with parameters c_p and a_p, minimum and maximum head requirements for the area based on the local fire and health standards (h_{min} and h_{max}), price of power (P_t) in $/kw-hr, historical average of maintenance costs for pumps from manufacturer to derive cost per switch ($/switch). The demand pattern (Pat_t) for the study area is needed.

3.6. Example Application

The model was applied to an example pumping station. Figure 3-1 shows an observed pump schedule for the study area, which is a realistic hypothetical pump schedule based on the state of art in practice. The model inputs for the example application are described here. The price of power (P_t) throughout a day in $/kw-hr is

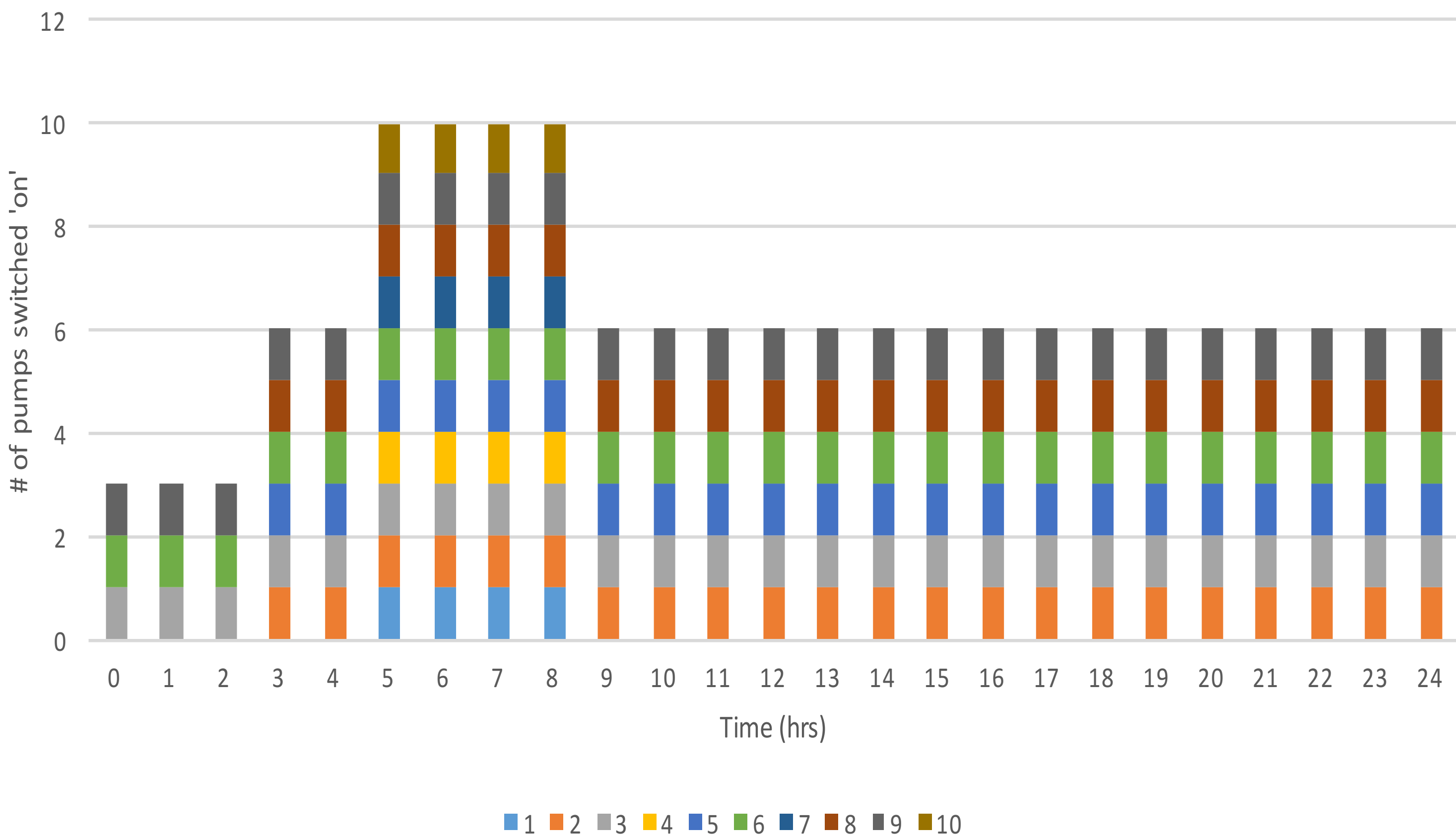


Figure 3-1 Observed Pump Schedule

given in Figure 3-2. The pumping station considered in this study consists of 10 pumps with three different types of pumps. The coefficients of the assumed pumps are listed in Table 3-1 for the respective pumps. A range of maximum switches allowed was used for the implementation.

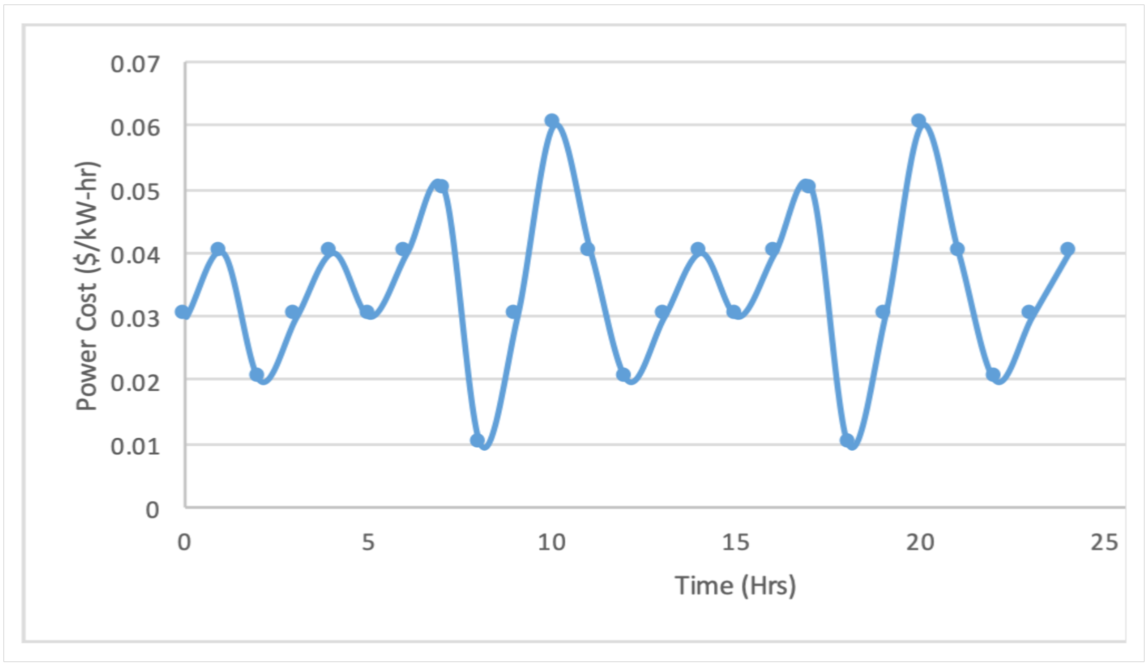


Figure 3-2 Trends of Prices of Power

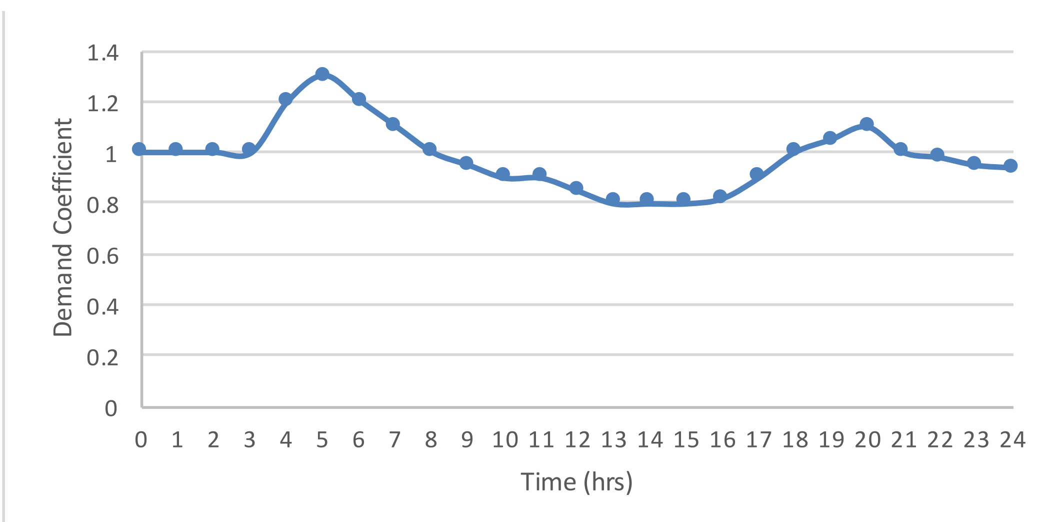


Figure 3-3 Demand Pattern

Table 3-1 Pump Coefficients

Pump	c_p	a_p
1, 2, 3 & 10	0.0078	218.1
4, 5 & 6	-0.01	182.18
7, 8 & 9	-0.0072	179.42

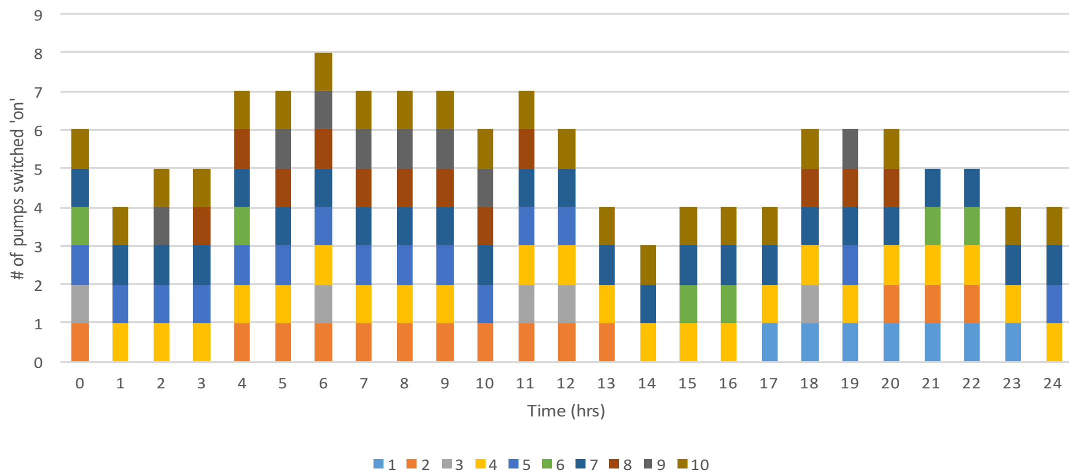
Total demands are met on an hourly basis. A Q_{prod} of 55,000 gpm was used as the base demand. Figure 3-3 shows the daily demand pattern for the WDS.

3.7. Application Results

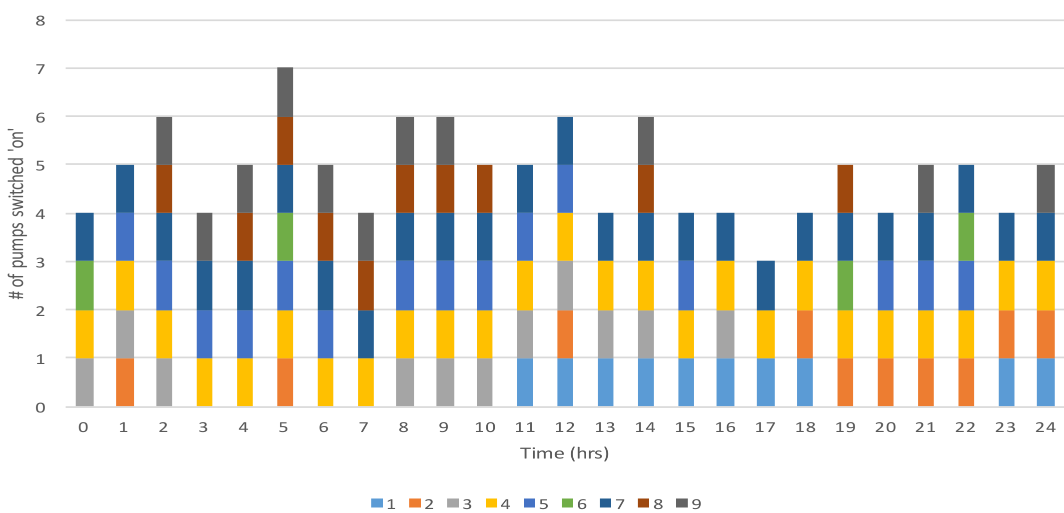
Figure 3-4 (a) shows the optimal pump schedule for no uncertainty in demands. Figure 3-4 (b - e) show the optimal pump schedules for uncertain demands with demand variations of 10%, 20%, 30% and 40% respectively. The number of pumps operating during the peak hours of energy consumption is considerably less in the optimized results as compared to the assumed observed data. The pump schedules are considerably different in case of the different variations of demand satisfaction considered in this study. The peak pumping in these cases is lower as compared to the observed data, since the model considers that there could be a discrepancy in demand satisfaction. Table 3-2 is a list of the operation costs for different demand variations along with the values of mean (μ) of Q_{prod} , the standard deviation (σ) and the cumulative distribution function (z) used for computation of the operation costs. Figure 3-5 shows the trends of operation costs for different demand variations.

Table 3-2 Trends of Daily Operation costs ($) with respect to Demand Variations

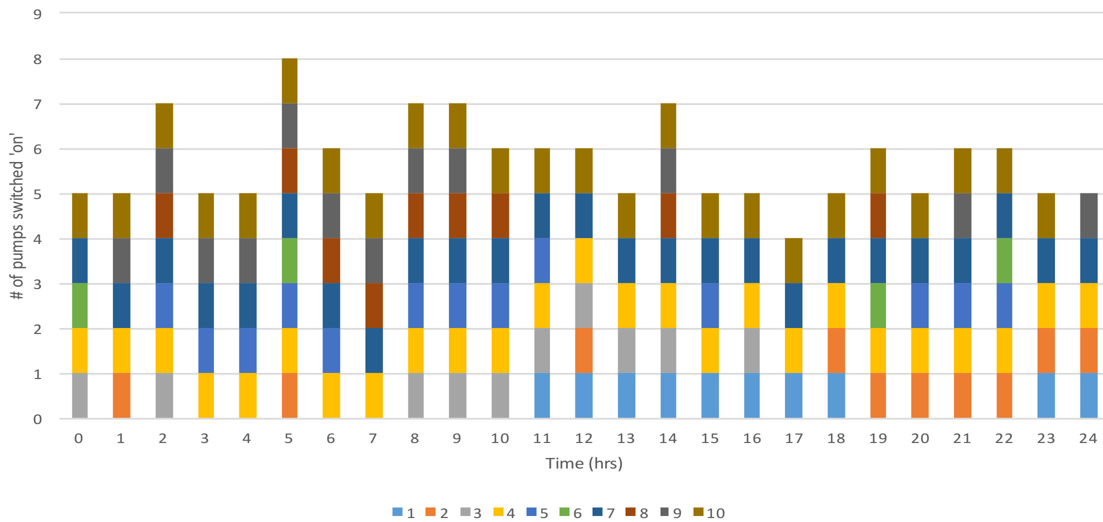
Demand Variation (%) $(1-\alpha)$	Mean (μ) (gpm)	Standard Deviation (σ) (gpm)	Daily Operation Cost ($)
10	53988	911.94	2352
20	53988	911.94	2340
30	53988	911.94	2326
40	53988	911.94	2313



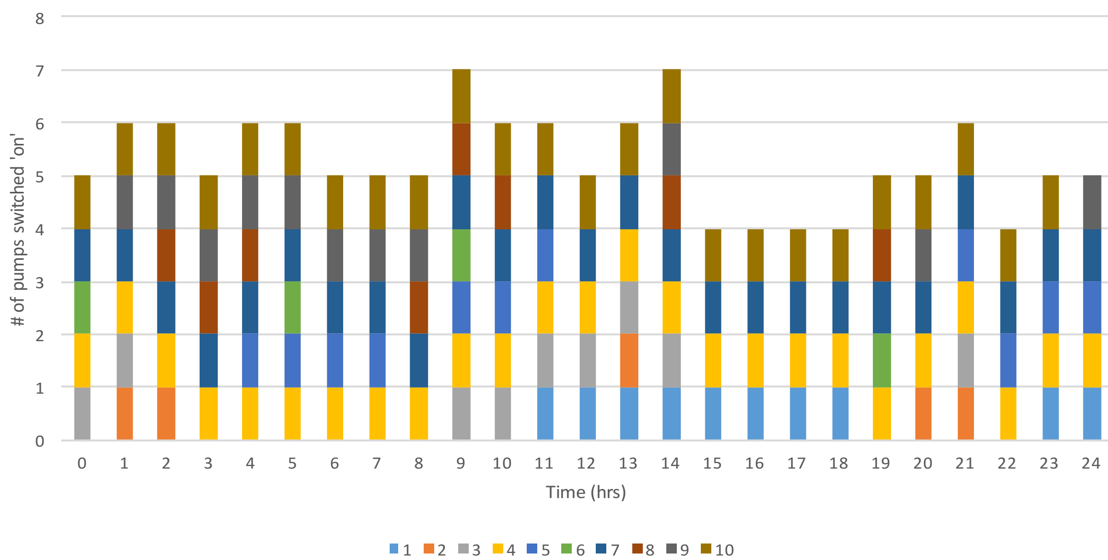
(a) No uncertainty



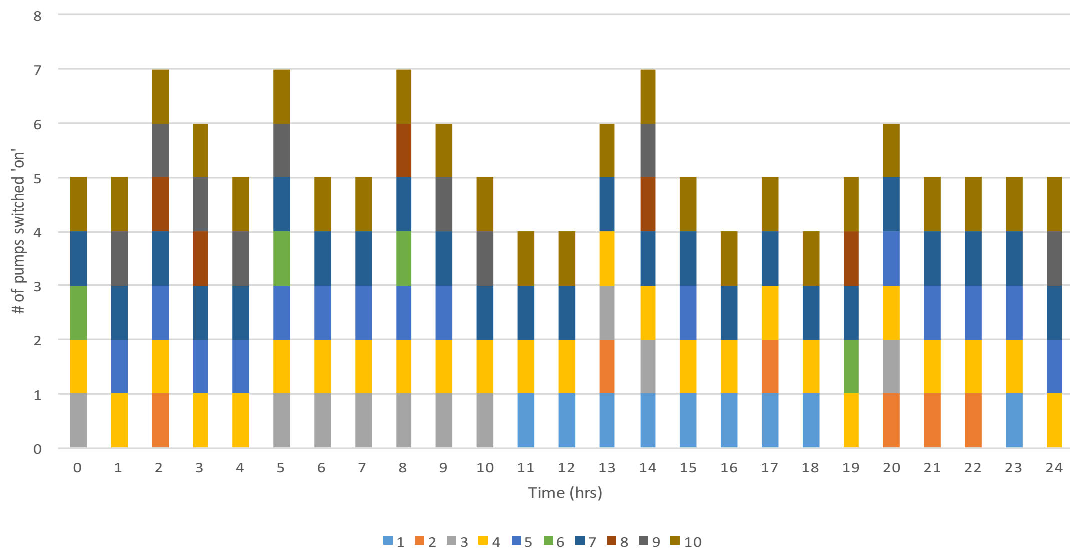
(b) 10% variation



(c) 20% variation



(d) 30% variation



(e) 40% variation

Figure 3-4 Optimal Pump Schedule for Uncertain Demands

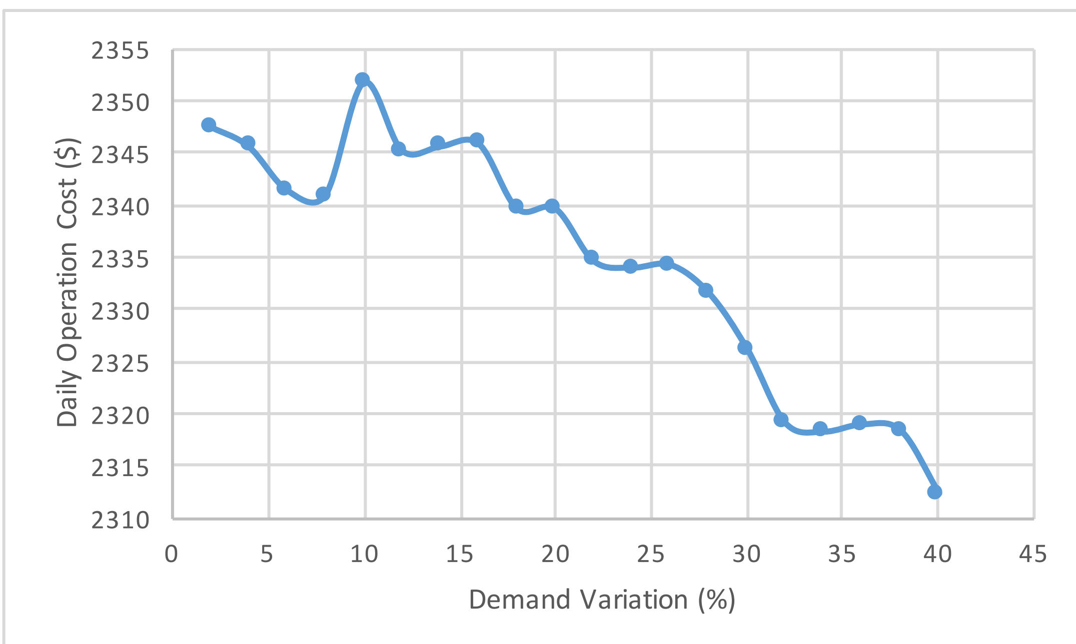


Figure 3-5 Operation Cost ($) Trends for Different Demand Variations

Table 3-3 lists the results of the application of the model for different variations of number of switches allowed, while Figure 3-6 shows the trends of the operation costs with respect to variations of number of switches allowed.

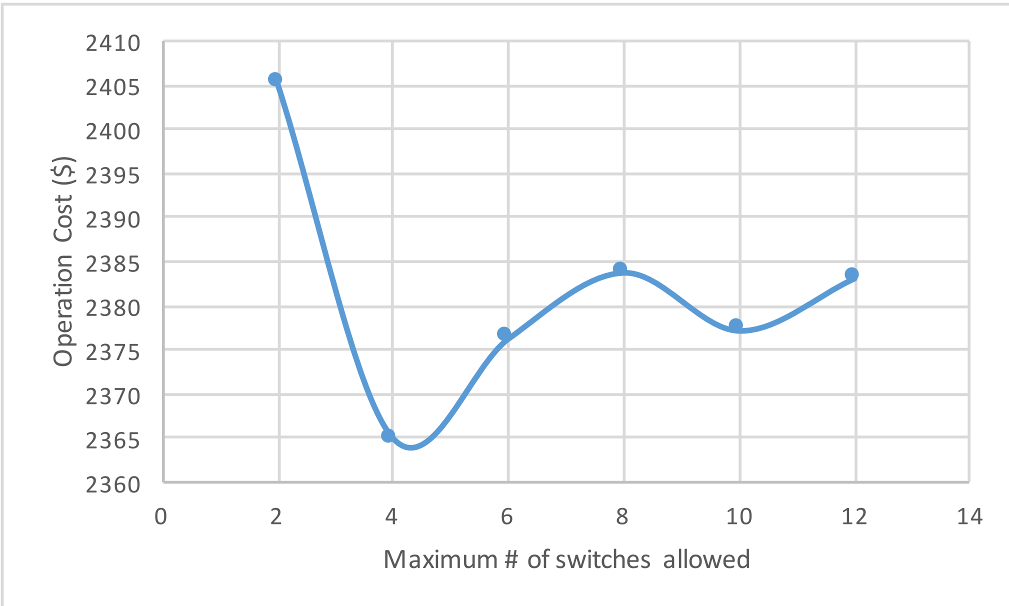


Figure 3-6 Operation Cost ($) Trends for Number of Switches Allowed

Table 3-3 Variation of Operation Costs with Maximum Number of Switches Allowed

Maximum no. of switches allowed	Operation Cost ($)
2	2405
4	2365
6	2376
8	2384
10	2377
12	2383

The models presented in this paper consider a constant average efficiency of pumps that are typically based on the various tests performed by the manufacturers and utilities. Inclusion of variable efficiencies for pumps is a major future change that could be incorporated in the model.

3.8. Observations

Figure 3-1 illustrates that several pumps are operated throughout the 24-hour period. This is a realistic pump schedule for the hypothetical water distribution system. The price of power varies throughout the day as shown in Figure 3-2. The power costs follow a diurnal pattern, with morning and evening peaks similar to the trend of water demand given in Figure 3-3.

Figure 3-4 (a - e) show that the number of pumps being operated at each hour of the day go on decreasing as the demand satisfaction constraint is further relaxed (variation increased). The optimization model thus considers the allowable variation in demand and the operation costs of the pumps to provide an optimized pump schedule for a particular case of uncertainty as well as for a certain demand pattern. The pump schedules depict that the schedule remains the same for off-peak demands as for the optimized schedule for the same set of pumps. The deficit is observed in case of the peak demands and the hours of peak power rates. This shows that the model ensures that all the demands as far as possible within the constraints are fulfilled, while minimizing the operation costs.

Figure 3-5 indicates that for demand satisfaction variations up to about 40%, there is a steady decline in the operation costs, while there is a considerable increase in the operation costs in case of variations above 40%. This sudden increase in operation costs depict inefficiency in pump operations due to reduced demands (demand satisfaction being relaxed). In practice, a demand satisfaction deficit of more than about 30% may not be acceptable even in emergency conditions, since the operation of vital services may be hindered on account of such a deficit. The observed pump schedule given in Figure 3-1 has an annual cost of $994,820, while the optimized pump schedule for the same demands (certain demands) given in Figure 3-4 (a) has an annual cost of $857,750. This shows a saving of about 12% in operation costs of the pumping station.

Operation costs are affected by several factors including the specific pricing policies of a power utility which includes seasonal and hourly variations in the power prices. For this application, the operation costs of the pumping station vary considerably with a change in the number of maximum allowable switches as seen in Figure 3-6. For a larger number of allowable switches, the operation costs are comparatively lower. As discussed earlier, the maintenance costs are not easily determined and hence the number of switches is used as a surrogate measure for these costs. Hence, though a reduction in the operation costs is observed during operation of pumps with an increased switch limit, maintenance costs may increase with such an increase in the number of switches.

3.9. Conclusions

A new methodology for generation of optimized pump schedules for certain and uncertain demands has been developed. This model, if implemented for an urban water distribution pumping system could result in considerable reduction in water supply expenses.

Since the methodology also aims at reduction of the maintenance costs by constraining the number of switches, the useful life of the pumping system is expected to enhance considerably. The optimized pump schedules, show that the number of pump switches (pumps being switched on and off) are reduced considerably as compared to the observed pump schedule. This considerably reduces the maintenance costs for the pumps, since switches are mainly responsible for the wear and tear of the pumps and are considered as a surrogate measure for the pump maintenance costs in this study.

This method can be applied to operations of a medium to large-scale municipal water distribution pumping station. Such an application of the methodology provides an optimized pump schedule for the system. The various requirements for the water distribution network such as tank levels and pressure requirements are taken into consideration in the head constraints. This provides a simple methodology for real-time control and decision making for pumping facilities of small to large-scale water distribution systems while considering the uncertainty in demand satisfaction, which is inherent in any real water distribution system.

A future step that could improve the model considerably is the reduction of the data prepossessing required for the inputs to the model. This could be achieved by incorporating new methodologies to use real-time data in the model from a SCADA system. Another important addition to the present approach would be to use the discharge-efficiency curves to define the efficiencies of the pump as an alternative for the current average efficiency approach.

An important addition to the solution strategy would include interface of the optimization model with the hydraulic solver EPANET, to compute the various hydraulic parameters for all the nodes in the network as an alternative to the equivalent system used in this paper.

4. MODEL FOR THE REAL-TIME OPERATIONS OF WATER DISTRIBUTION SYSTEMS UNDER LIMITED ELECTRICAL POWER AVAILABILITY WITH CONSIDERATION OF WATER QUALITY

4.1. Summary

A new methodology is presented for real-time operation of water distribution systems (WDS) under the critical condition of limited electrical energy. The critical conditions could arise due to electric grid failure, extreme drought or other severe conditions related to natural and manmade disasters such as sabotage, vandalism, terrorism or war. The methodology presented considers both quantity and quality requirements of various water demands. The basic objective of optimizing water distribution system operations under limited availability of electrical power and/or water is to satisfy the required (requested) demand for service areas (or pressure zones) while meeting system pressure and water quality requirements of the system. The approach interfaces a genetic algorithm optimization procedure with the simulator EPANET in the framework of an optimal control problem. Interfacing of the simulator and the genetic algorithm is accomplished within a MATLAB framework. The new methodology is illustrated using an example system incorporating both a WDS and electrical power distribution system (PDS) cooling water system to evaluate the operations of the WDS under limited power supply conditions.

4.2. Introduction

The interdependencies between the electric power distribution systems (PDS) and water distribution systems (WDS) have long been recognized. These two systems, collectively, are referred to as the water-energy nexus. The main dependency of the WDS on the PDS is for electrical power for pumping and the main dependency of the PDS on the WDS is for required water in the cooling cycle. The interaction between these two critical infrastructures are studied using a coupled, time-domain simulation. The methodology presented in this paper is based on research conducted for a US National Science Foundation (NSF) Project 029013-0010 titled ‘CRISP Type 2 – Resilient cyber-enabled electric energy and water infrastructures modeling and control under extreme drought’. The model simulates real time data exchanges between the PDS and WDS using Software Defined Network (SDN) middleware architecture to allow the study of how the linked systems respond to power failures, drought or other stress conditions (see Figure 4-1). This overlay enables a reliable and efficient data exchange between the two otherwise isolated systems. This is a representation of how the two systems are operated when subjected to various disturbances in either system as well as under conditions of long-term water and power shortages.

The decisions regarding operation of a WDS include hourly operation of pumps throughout the day based on demand variation and electrical power availability. An efficient operation of these essential components of the system facilitates appropriate functioning of the system for supply of water in required quantity and quality to the consumers. The methodology presented uses hourly pump control to operate the water distribution system (WDS) under conditions of limited power availability while optimizing compliance with hydraulic conditions, system demand and water quality constraints.

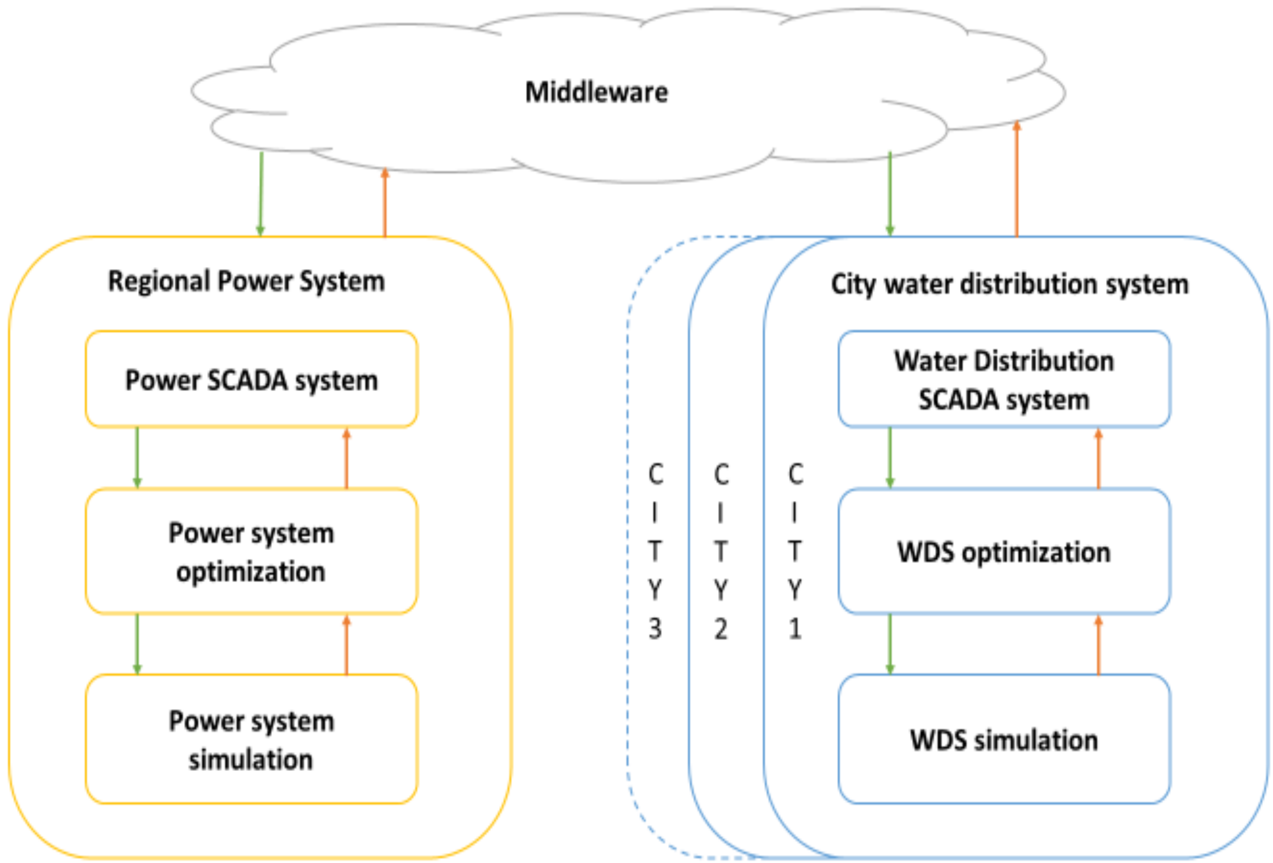


Figure 4-1 Real-time Operation Model for Power and Water Distribution Systems Operation

The model development here simulates both the hydraulics and the water quality in the WDS and minimizes the differences between demands required and the satisfied demands by varying pump operation.

4.3. Previous WDS Optimization Models

A few attempts for optimizing the operations of WDS to control the water quality within a WDS are reported in the literature. Burn and McBean (1985) presented a stochastic modeling approach for water quality parameters in a WDS. Beck (1987) used a heuristic approach for modeling the water quality parameters to explain the role of uncertainty in water quality prediction. Maier and Dandy (1996) discussed the use of artificial neural networks for the prediction of water quality parameters in a WDS.

Sakarya and Mays (2000) presented a non-linear programming methodology for determining optimal operation of WDS pumps to meet water quality considerations. The solution methodology used in the study included an interface between the hydraulic simulator (EPANET) and a nonlinear optimization code, GRG2. Sakarya et al. (1999) and Goldman et al. (2002) explained a non-linear optimization and a simulated annealing model interfaced with EPANET for optimizing pump schedules considering the water quality requirements in a WDS. Trawicki et al. (2003) presented a hybrid genetic algorithm (GA) approach for optimizing quality and quantity of water in a WDS.

Goldman and Mays (2005) developed a methodology that linked a simulated annealing model with EPANET for obtaining optimal pump operations for a WDS while meeting both quality and hydraulic performance requirements. One of the more recent works for optimization of operations of water distribution systems considering quality requirements was by Kurek and Ostfeld (2014). They used a strength pareto evolutionary algorithm (SPEA2) methodology for satisfying quality and storage-reliability constraints. The model considers pumping cost, water quality and storage reliability constraints while optimizing the pump controls in the system. Van Blaricum et al. (2016) demonstrated the use of a model for monitoring water quality and corrosion in a WDS considering real-time operations of the WDS and an interface with a SCADA system.

4.4. Real-time Operation Framework During Limited Electrical Power Generation

The basic objective of optimizing the operation of a water distribution system under limited availability of electrical power is to satisfy the required levels of demand at various locations while also meeting pressure requirements of the system. The objective statement formulated here is to minimize the differences between the required demand and the satisfied demands in all service areas (or pressure zones) in the system. The following describes the overall concept of the real-time operation for each municipal water system:

1. At real-time t receive electrical power input including power availability at pumps and required water demand multipliers for pumps from PDS.
2. Receive latest data (status of pumps, tank levels, status of valves, and flows in and out of the system from the SCADA system for each water distribution system (WDS).
3. Update the EPANET model for each water distribution system (WDS) input using data from the SCADA system for WDS.

4. WDS optimization model is run to determine the demand pattern that can be met and pump operation which satisfy the limited electrical energy constraint. During the optimization/simulation process, the EPANET model is called repeatedly from the genetic algorithm optimization to determine WDS operations over the next 24 hours. The optimization model searches over the decision variables (pump operation and satisfied demand pattern) to minimize the difference between demand required and demand satisfied. The simulator determines the values of the state variables (nodal pressure heads and tank levels) for each set of control variables (pump operations and satisfied demand pattern) determined in the optimizer.
5. Implement the optimal pump schedule over the next hour which is accomplished through the SCADA system.
6. Repeat steps 1 through 5 each time increment during the emergency event incrementing the time $t = t + \tau$ with $\tau = 1$ hour.

4.5. Modeling Approach

Khatavkar and Mays (2017 a and b) briefly presented preliminary optimization - simulation models for real-time operations of a water distribution system under conditions of limited power availability. These models consider the pressure and demand requirements of the WDS but does not consider water quality requirements. The preliminary optimization model presented by Khatavkar and Mays (2017 a and b) has been significantly modified and extended to include the water quality constraints.

Consider a WDS with Z service areas (or pressure zones), K nodes, M pipes, S tanks and Pu pumps for a simulation time of T hours.

4.5.1. Objective

The objective of the optimization model for operations of WDS presented in this paper is to minimize the difference between the required demand ($D_{req,\,z,t}Qb_z$) and the satisfied demand ($D_{sat,\,z,t}Qb_z$) for all the service areas (z) (or pressure zones) and simulation time (t).

The objective function is as follows

$$\min \mathrm{Obj} = \sum_{t}^{t=T} \sum_{z}^{z=Z} [(D_{req,\,z,t} - D_{sat,z,t})Qb_z]^b \theta_{z,t} \tag{4-1}$$

Where, Obj is the objective function; $D_{req,\,z,t}$ and $D_{sat,\,z,t}$ are the required and satisfied demand multipliers respectively for service area (or pressure zone) z at time t; b is the power index for objective function; Z is the total number of service areas (or pressure zones) in the system; T is the total simulation time; Qb_z is the average demand (gpm) observed during 24-hour operations of the WDS for service area (or pressure zone) z and $\theta_{z,t}$ is a penalty multiplier for not satisfying demand for service area (or pressure zone) z at time t.

4.5.2. Constraints

1. Equations solved implicitly by EPANET for hydraulic analysis.

The distribution of flow throughout the network must satisfy the conservation of mass and the conservation of energy which are defined as the hydraulic constraints. The conservation of mass at each junction node, assuming water is an incompressible fluid, is

$$\sum_i (Q_{i,k})_t - \sum_j (Q_{k,j})_t - Q_{k,t} = 0 \qquad \forall k = 1, \ldots, K \text{ and } t = 1, \ldots, T \tag{4-2}$$

where $(Q_{i,k})_t$ is the flow in the pipe m connecting nodes i and j at time t (gpm) and Q_{kt} is the flow consumed (or supplied) at node k at time t (gpm); K is the total number of nodes in the system.

The conservation of energy for each pipe m connecting nodes i and j, in the set of all pipes, M is,

$$H_{i,t} - H_{j,t} = f(Q_{i,j})_t \qquad \forall\ i, j \in K \text{ and } t = 1, ..., T \qquad (4\text{-}3)$$

$H_{i,t}$ and $H_{j,t}$ are the pressure heads at nodes i and j respectively.

The total number of hydraulic constraints is (K+M) T, and the total number of unknowns are the discharges in M pipes and the hydraulic grade line elevations at K nodes. The depth of water stored in a tank (s) for the current time period ($y_{s,t}$) is a function of the depth of water stored from the previous time period are

$$y_{s,t} = f(y_{s,t-1}) \qquad \forall s = 1, ..., S \text{ and } t = 1, ..., T \qquad (4\text{-}4)$$

The bounds on the level of water storage in a tank s for time t are

$$\underline{y}_{s,t} \leq y_{s,t} \leq \bar{y}_{s,t} \qquad \forall s = 1, ..., S \text{ and } t = 1, ..., T \qquad (4\text{-}5)$$

where $\underline{y}_{s,t}$ and $\bar{y}_{s,t}$ are the lower and upper bounds, respectively of the depth of water stored in node s at time t, $\bar{y}_{s,t}$. These limits are normally due to physical limitations and fire flow storage requirements of the storage tank.

2. Equations solved implicitly by EPANET for water quality analysis (Rossman and Boulos. 1996; Rossman, 2000). The EPANET water quality simulation is based on the discrete volume method (Rossman et al. 1993).

3. The bounds on pump operation time are given as

$$\Delta t_{min,p} \leq \tau \sum_t X_{p,t} \leq \Delta t_{max,p} \qquad \forall p = 1, ..., Pu \text{ and } t = 1, ..., T \qquad (4\text{-}6)$$

Where $\Delta t_{min,p}$ and $\Delta t_{max,p}$ are lower and upper bounds respectively on total number of hours pump p is switched on; τ is the time interval for WDS simulation and $X_{p,t}$ is a binary variable for pump operations (1 refers to pump p "switched on" at time t and 0 refers to pump p "switched off" at time t).

4. The 'switch' constraint

A switch means turning on or off of the pump. The following constraint limits the number of switches of a pump during WDS operations.

$$Sw_{min(p)} \leq \sum_t \max(0, (X_{p,t} - X_{p,t-1})) \leq Sw_{max(p)} \quad \forall p = 1, ..., Pu \text{ and } t = 1, ..., T \tag{4-7}$$

Where $Sw_{min(p)}$ and $Sw_{max(p)}$ are the minimum and maximum number of switches allowed for pump p respectively.

5. Power constraint

The power requirement of the pumps in the system are computed using the equation given by Khatavkar and Mays (2017 c). An upper bound on the power use for pump p and time t is given by the power available ($P_{Avail(p,\ t)}$).

$$X_{p,t}\frac{Q_{p,t}\,H_{p,t}}{3956\eta_p} \leq P_{Avail(p,\ t)} \qquad \forall p = 1, ..., Pu \text{ and } t = 1, ..., T \tag{4-8}$$

Where $Q_{p,t}$ is the flow from pump p at time t (gpm) and $H_{p,t}$is the pressure head provided by pump p at time t (ft).

6. The nodal pressure head bounds (including tank levels) are expressed as

$$\underline{H}_k \leq H_{k,t} \leq \overline{H}_k \qquad \forall p = 1, ..., Pu \text{ and } t = 1, ..., T \tag{4-9}$$

Where $H_{k,t}$ is the pressure head and $\underline{H}_k$ and $\overline{H}_k$ are upper and lower bounds for pressure head for node k and time t.

7. Demand satisfaction constraint

The bounds on satisfied demand multipliers ($D_{sat,z,t}$) are given as

$$a\, D_{req,\, z,t} \leq D_{sat,z,t} \leq D_{req,\, z,t} \qquad \forall z = 1, \ldots, Z \text{ and } t = 1, \ldots, T \tag{4-10}$$

Where a is a multiplier for minimum demand requirement ($a \leq 1$).

7. Water quality constraint

Several contaminants/nutrients may be required to be monitored and controlled during operations of a WDS. This optimization methodology attempts to keep the concentration of the various monitored contaminants/nutrients within a range of a lower and upper bound. The constraint for concentration ($C_{n,z,t}$) of a nutrient/contaminant n at time t and service area z is given as follows.

$$\underline{C}_n \leq C_{n,z,t} \leq \overline{C}_n \qquad \forall n = 1, \ldots, N;\ z = 1, \ldots, Z \text{ and } t = 1, \ldots, T \tag{4-11}$$

Where $\underline{C}_n$ and $\overline{C}_n$ are upper and lower bounds on concentration of nutrient/contaminant n respectively.

4.6. Reduced Optimization Model

The above optimization model is an optimal control problem solved by interfacing an optimization model (genetic algorithm) with the EPANET simulator. The genetic algorithm solves an unconstrained problem which is a reduced optimization problem composed of equations 4.1 and 4.6 to 4.11 and the simulator (EPANET) solves equations 4.2 to 4.5 implicit to the optimization model each time they are called. Figure 4-2 shows the overall framework for solving the optimization-simulation model.

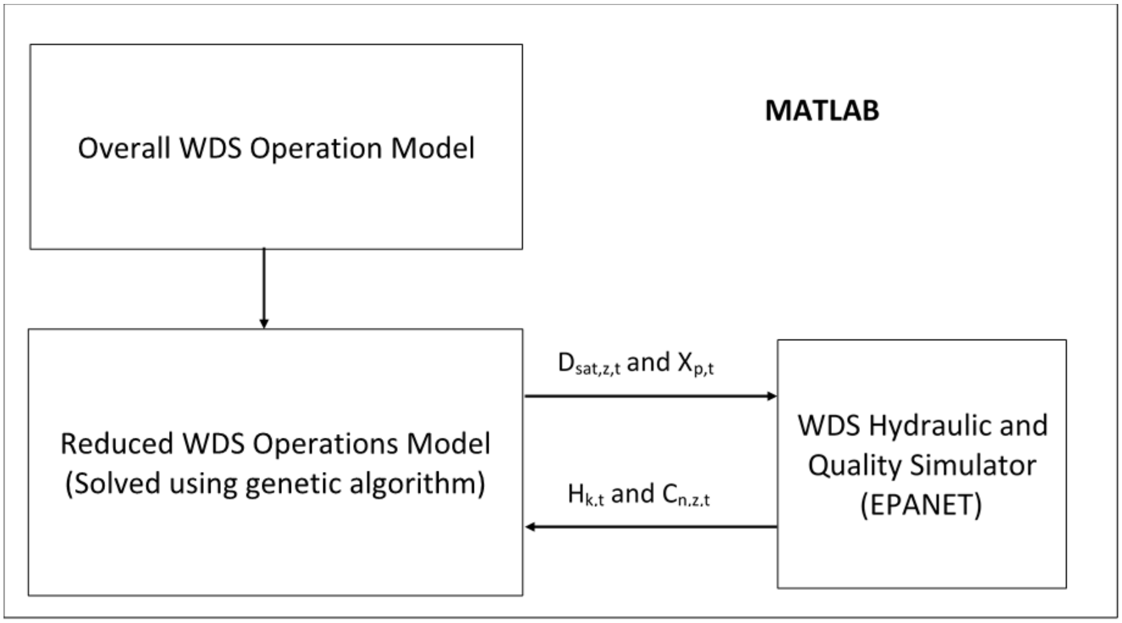


Figure 4-2 Optimization Framework for WDS Operation

A reduced optimization model with constraints (equation 4.6 – 4.11) in the form of penalty functions is solved by the genetic algorithm which solves unconstrained problems. The reduced optimization model is as follows:

Minimize $\mathrm{Obj}_{reduced} =$

$$W_{1,t}\sum_t^{t=T}\sum_z^{z=Z}\left[\left(D_{req,\,z,t} - D_{sat,z,t}\right)Qb_z\right]^b \theta_{z,t} + W_{2,t}\sum_t^{t=T}\sum_k^{k=N}\left(w_1\left(\max\left(0,\left(H_{k,t} - \overline{H_k}\right)\right)\right)^2 +\right.$$

$$w_2\left(\min\left(0,\left(H_{k,t} - \underline{H_k}\right)\right)\right)^2 + w_3\left(\min\left(0,\left(H_{k,t} -\right.\right.\right.$$

$$\left.\left.\left.0\right)\right)\right)^2\Big) + W_{3,t}\left[\sum_p^{p=Pu}\left(\max\left(0,\left(\Delta t_{min,p} - \tau\sum_t^{t=T}X_{p,t}\right)\right)\right)^2 +\right.$$

$$\left.\sum_p^{p=Pu}\left(\max\left(0,\left(\tau\sum_t^{t=T}X_{p,t} - \Delta t_{max,p}\right)\right)\right)^2\right] + W_{4,t}\left[\sum_p^{p=Pu}\left(\max\left(0,\left(Sw_{min(p)} -\right.\right.\right.\right.$$

$$\left.\left.\left.\sum_t \max\left(0,\left(X_{p,t} - X_{p,t-1}\right)\right)\right)\right)\right)^2 +$$

$$\sum_{p}^{p=Pu}\left(\max\left(0,\left(\sum_{t}\max\left(0,\ \left(X_{p,t}-X_{p,t-1}\right)\right)-Sw_{max(p)}\right)\right)\right)^{2}\ \Bigg]+$$

$$W_{5,t}\sum_{t}^{t=T}\sum_{p}^{p=Pu}\left(\max\left(0,\left(X_{p,t}\frac{Q_{p,t}\,H_{p,t}}{3956\eta_{p}}-P_{Avail(p,t)}\right)\right)\right)^{2}$$

$$+W_{6,t}\sum_{n}^{n=N}\sum_{t}^{t=T}\sum_{z}^{z=Z}\left(wc_{1}\left(\max\left(0,\left(C_{n,z,t}-\overline{C_{n}}\right)\right)\right)^{2}+wc_{2}\left(\min\left(0,\left(C_{n,z,t}-\underline{C_{n}}\right)\right)\right)^{2}\right)$$

(4-12)

where $W_{1,t}$-$W_{6,t}$ are penalty weights associated with demand satisfaction, pressure constraints, power constraints, total power utilization, switch constraints and water quality constraints, respectively; Pu is the total number of pumps in the system and N is the total number of contaminants/nutrients being monitored.

The penalty weights ($W_{1,t}$-$W_{6,t}$) are determined through a sensitivity analysis for each application of the optimization model. The sensitivity analyses are performed for all terms involved in the reduced optimization model. The value of weights to be used for a particular term depend upon the relative importance of the penalty term and the magnitude of the numeric value of the penalty term. Parameters considered for the sensitivity analyses are the value of the objective function, number of pressure bound violations, number of power bound violations and the number of water quality bound violations. The optimization model is tested for a range of values for each of the penalty weights and a combination of penalty weights yielding the minimum objective function value and least amount of bound violations for pressures, power and water quality is chosen for the application.

The control variables for this model are the satisfied demand pattern ($D_{sat,z,t}$) and the pump operations ($X_{p,t}$), a binary variable defining if a pump is switched on ($X_{p,t} = 1$)

or off ($X_{p,t} = 0$). The state variables for this optimal control problem are the nodal pressure heads (including tank levels) ($H_{k,t}$) and the water quality at nodal locations ($C_{n,z,t}$). A penalty is applied for infeasible solutions returned by EPANET using the penalty weight (w_3) for negative pressures in the reduced optimization model given in equation 4.12.

4.7. Optimization - Simulation Model Interface

The overall model interface between the genetic algorithm (WDS optimization model) and the EPANET simulator is accomplished using MATLAB as shown in Figure 4-2. The interface between MATLAB and EPANET is performed using the open-source EPANET-MATLAB toolkit (Elíades, 2009). The interface facilitates use of all the functionalities of EPANET within the MALTAB environment by passing the various commands to and from the MATLAB mathematical language to the EPANET simulator. This toolkit was also used in conjunction with the genetic algorithm (GA) in MATLAB to accomplish an overall optimization/ simulation methodology. The methodology works through an interface facilitating data exchange between the genetic algorithm and EPANET as shown in Figure 4-2.

The WDS optimization – simulation model is run for an extended period of time. Starting with the time-period, the WDS optimization-simulation model receives inputs from the PDS model including observed and predicted power availability schedule for each time-period. The demands from the power plants are communicated to the WDS optimization – simulation model on an hourly basis from the PDS. The WDS optimization - simulation model is then run with the updated information to obtain the pump controls and satisfied demand multipliers for the next time-period, several times until the GA stopping conditions are met. The solution of the GA is transmitted to the PDS optimization-simulation model. This process is continued until the last simulation time-period is reached.

4.8. Example System

The basic purpose of this example is to illustrate the optimal operation under power outages and the consideration of with and without TDS. The example system is a hypothetical WDS (Figure 4-3), including the WDS's for two cities and a PDS with five power plants based on the IEEE 14 bus system (Kodsi and Canizares, 2003). The cooling water for these power plants is supplied from both a freshwater source and a reclaimed wastewater source (waste water treatment plant (WWTP) shown in Figure 4-3). City 1 has four service areas (defined by nodes 1.1 – 1.4) with a total base demand of 30,000 gpm. City 2 has five service areas (defined by nodes 2.1 – 2.5) with a total base demand of 25,000 gpm. A total of 17 freshwater pumps and 11 reclaimed water pumps serve the overall WDS. Figure 4-4 shows the layout of the WDS within the two cities. Figure 4-5 shows the demand pattern multipliers for all the residential demands in the WDS. Cooling water demands for power plants, on an hourly basis, are input to the WDS optimization – simulation model from the PDS optimization – simulation model.

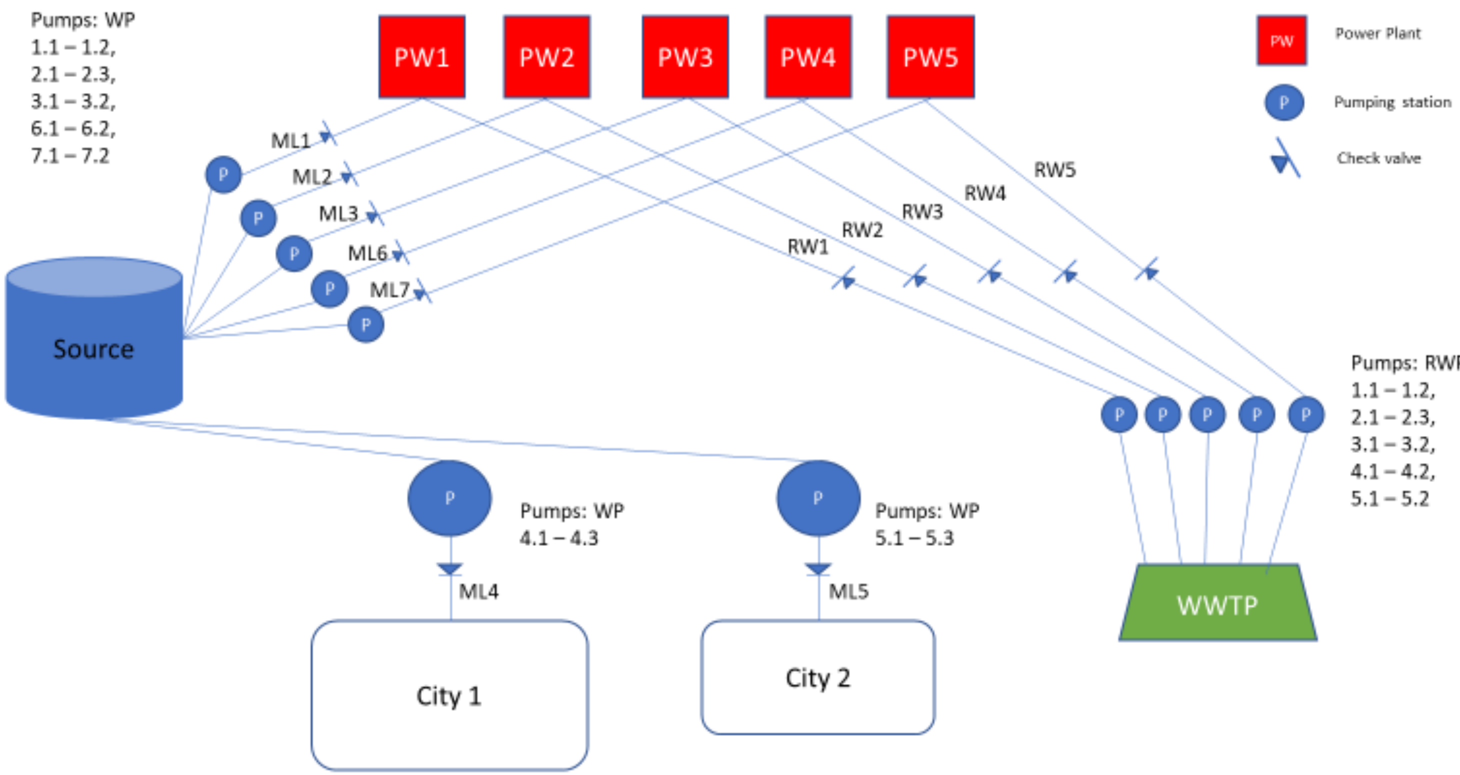


Figure 4-3 Schematic of Overall Water Distribution System (WDS)

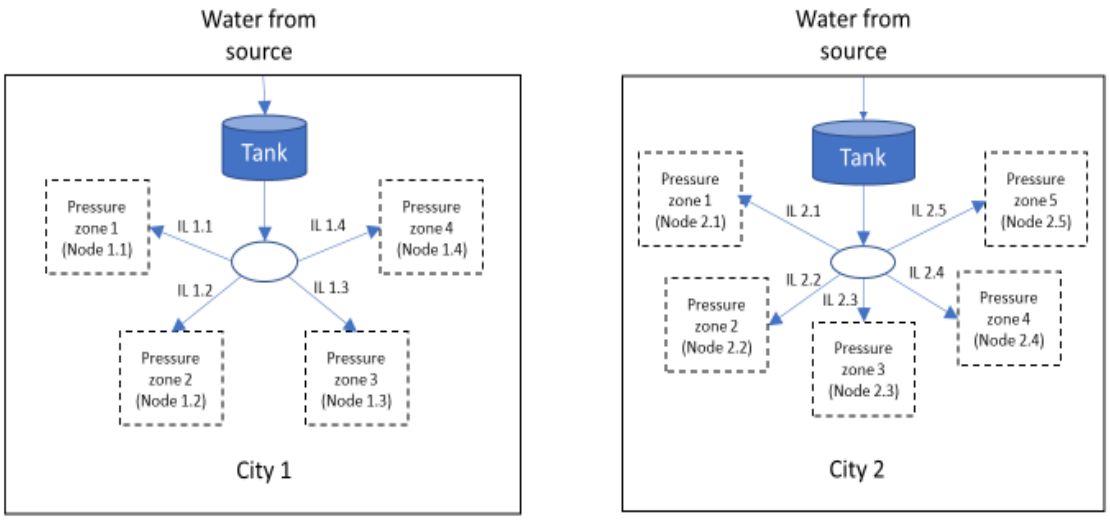


Figure 4-4 Water Distributions Systems (WDS) of City 1 and City 2

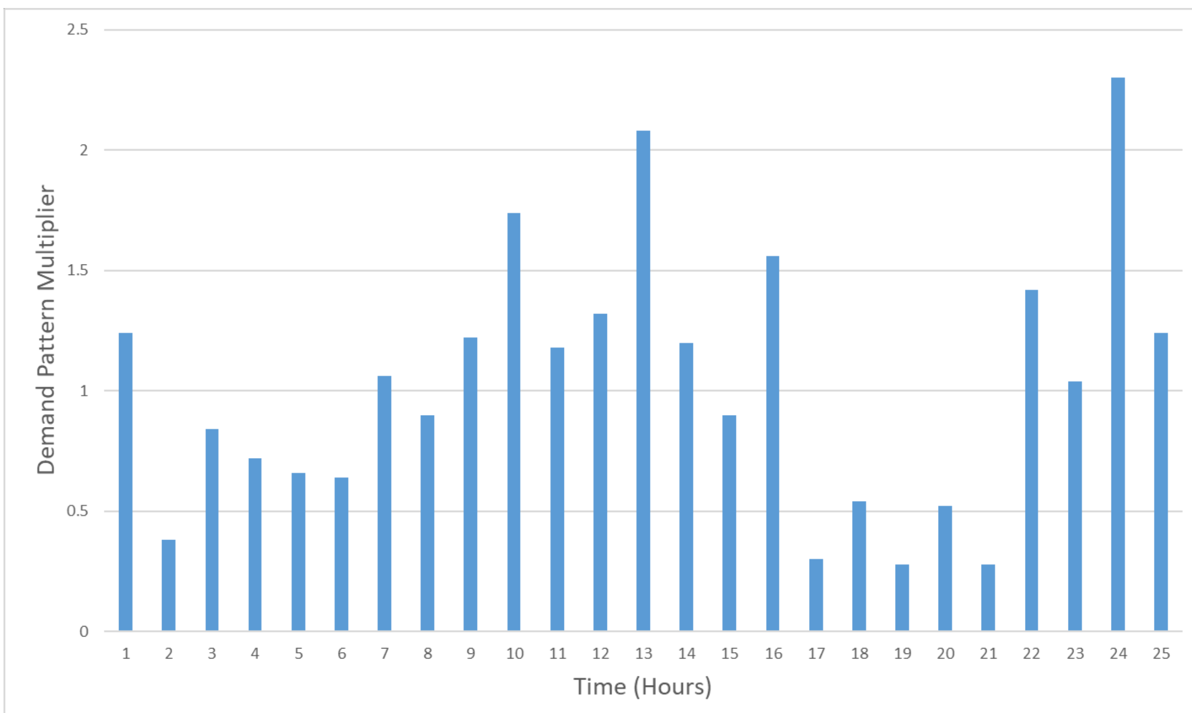


Figure 4-5 Demand Pattern for Residential Service Areas in Example WDS

Pipes in the WDS are categorized into main lines, intermediate lines, and reclaimed water lines. Main lines (ML1 – ML7) connect the freshwater pumps to the power plants and cities; intermediate lines (IL 1.1 – IL 1.4 and IL 2.1 – IL 2.5) connect nodes within the cities, and reclaimed water lines (RW1 – RW5) connect the waste water treatment plants to the five power plants in the system. The pumps supporting the overall WDS are categorized as fresh water pumps (WP1 – WP7 series) and reclaimed water pumps (RWP1 – RWP5 series).

Table 4-1 lists the schedule of power outages for each pump affected. The inputs for water quality analysis (TDS) in EPANET include initial source quality, source base concentration, time pattern for TDS concentrations, and reaction coefficients. TDS source concentration of 200 mg/l was used for the freshwater and 1500 mg/l for the reclaimed water. The reactivity of dissolved solids in the reclaimed water are negligible and hence no reaction coefficient was used. A common upper bound for the TDS in power plant cooling water is 1000 mg/l according to Choudhury et al. (2012).

Table 4-1 Details of Pumps Affected by Power Outages

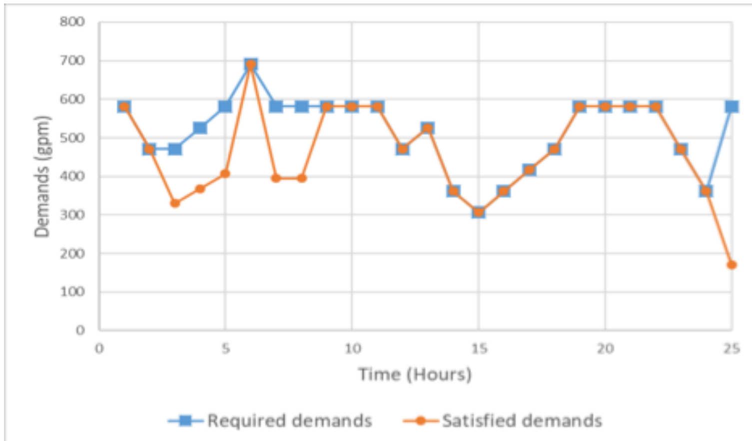
Pump #	Time of Power Outage (Hours)	Power Plant served
WP 3.1	3:00 to 5:00	PW3
WP 3.1	6:00 to 8:00	PW3
WP 3.1	10:00 to 12:00	PW3
WP 6.1	3:00 to 5:00	PW5
WP 6.1	6:00 to 8:00	PW5
WP 6.1	10:00 to 12:00	PW5
RWP 1.1	3:00 to 5:00	PW1
RWP 1.1	6:00 to 8:00	PW1
RWP 2.2	3:00 to 5:00	PW2
RWP 2.2	6:00 to 8:00	PW2

A sensitivity analysis was performed to determine penalty weights ($W_{1,t}$-$W_{6,t}$) of the reduced optimization objective (equation 4.12) for genetic algorithm. The penalty weights $W_{1,t}$-$W_{6,t}$ determined for the example system were 1 x 10^{-2}, 1.32 x 10^{5}, 1 x 10^{3}, 1 x 10^{4}, 1.28 x 10^{50} and 2.35 x 10^{4} respectively. The value of penalty weight associated with the power availability constraint (W_5) is highest due to the importance associated with the power availability bound.

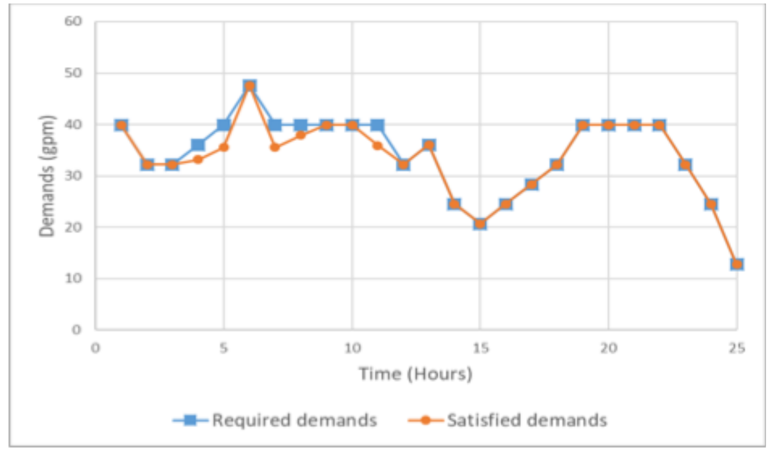
4.9. Example Application

The WDS optimization – simulation model was applied to the overall WDS system for two scenarios, each with power outages listed in Table 4-1. The first scenario considered power outages (shown in Table 4-1) without water quality (TDS) considerations and the second scenario considered TDS with a concentration upper bound of 1000 mg/l at the power plants. Demand deficits were observed at power plants PW1, PW3 and PW5. Figure 4-6 (a) – (c) show the trends of required and supplied demands for PW1, PW3 and PW5 for the scenario 1 application. The trends of required and satisfied demands were the same for both scenarios since the demand deficits observed were mainly due to the power outages.

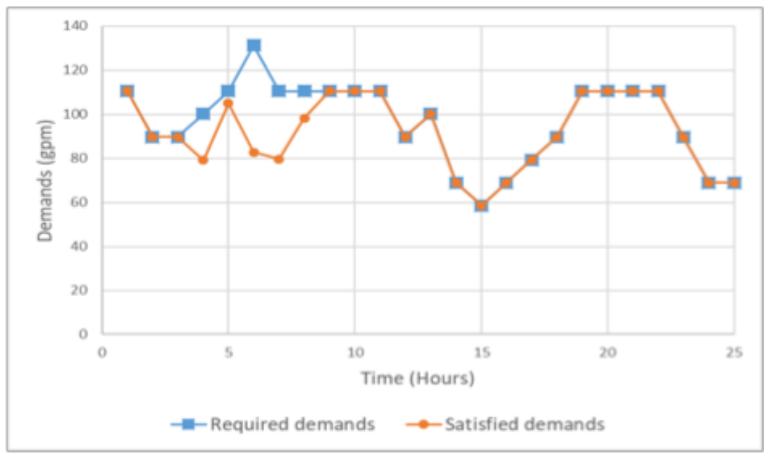
The demand deficit (difference between the required and supplied demands) were minimized in the system (see equation 4.12). The resulting demands shown in Figure 4-6 (a) – (c) depict minimal deficits between the required and supplied demands which is the objective (equation 4.12) of the optimization model. The average demand satisfaction was 95 percent for both the scenarios. The demand deficits are experienced during the period of power outages. Changes in the pumping schedule due to the power outage, leads to adverse effects on the water storage in the system as shown in Figure 4-6 (a) – (c).



(a) Demands for PW1



(b) Demands for PW3



(c) Demands for PW5

Figure 4-6 Required and Satisfied Demands for Affected Power Plants in the Example System

Scenario 2 considered a TDS concentration upper bound of 1000 mg/l for power plants in the WDS. Achieving a compliance of the allowable concentrations is one of the goals of this application. The power plants affected by the TDS concentrations are PW1 and PW2. Figure 4-7 shows the simulated and allowable total dissolved solids (TDS) concentrations for the power plants PW1 and PW2 in the system for scenario 2. Results in Figure 4-7 indicate that the simulated TDS concentrations do not exceed the upper bound of 1000 mg/l for scenario 2.

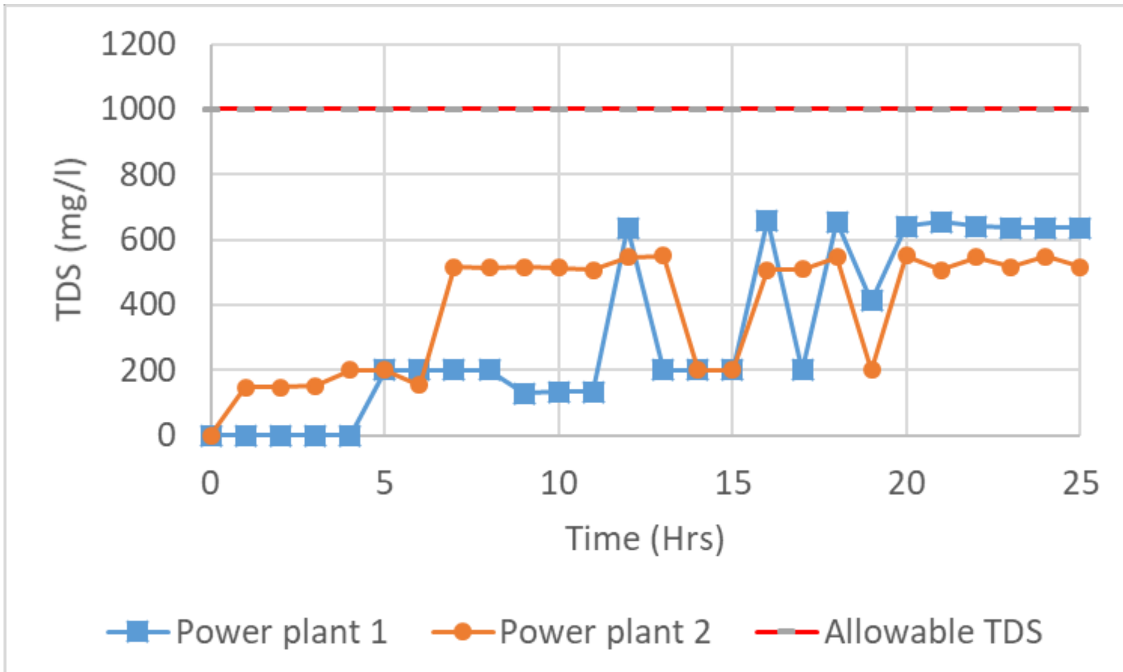
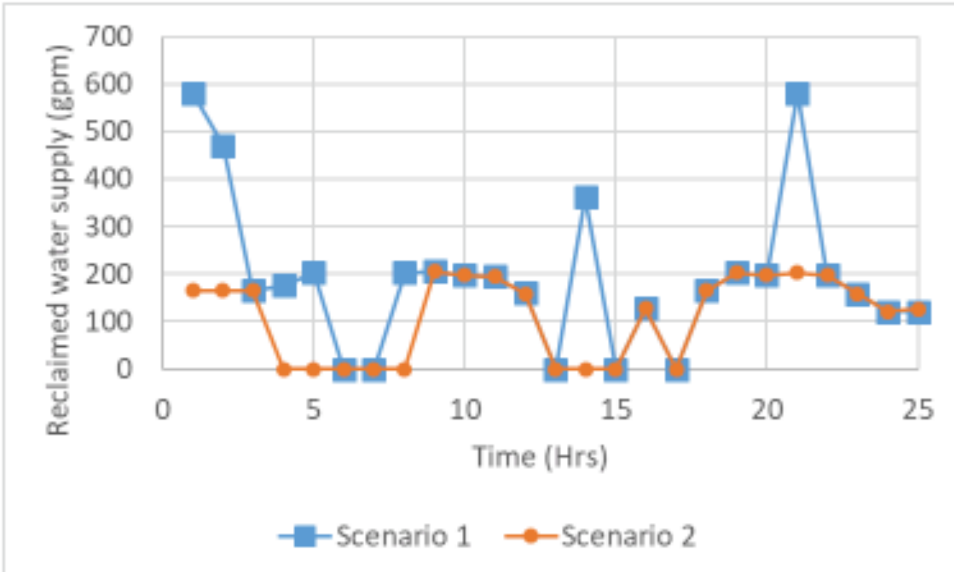
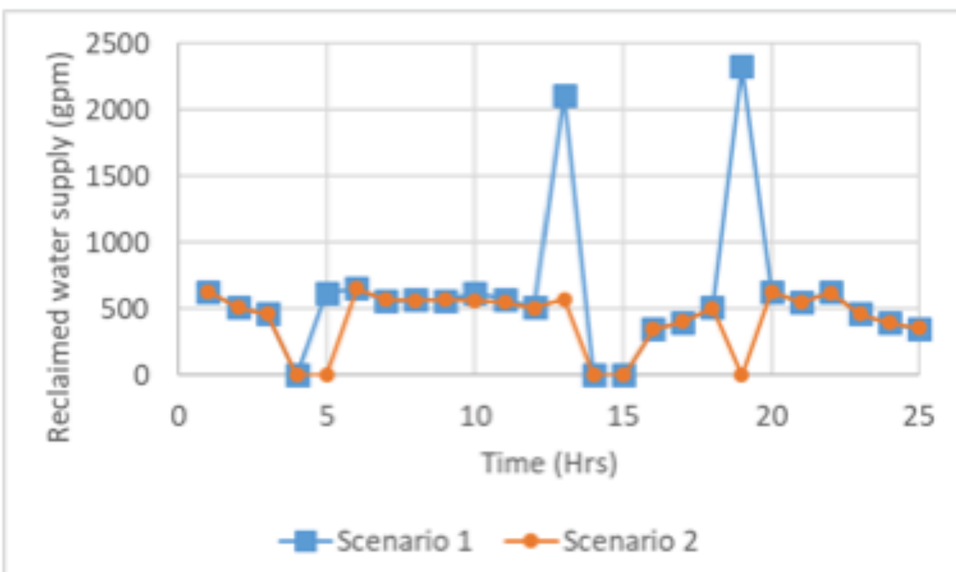


Figure 4-7 Trends of Simulated and Allowable TDS for Power Plants PW1 and PW2 for Scenario 2

Figure 4-8 (a) and (b) show the trends of reclaimed water supplied for power plants PW1 and PW2 respectively for both the scenarios with and without consideration of TDS. For scenario 2, the pump schedule was optimized within the GA to satisfy the upper bound on TDS. Figure 4-8 (a) and (b) indicate that reclaimed water supply was decreased in scenario 2 to satisfy the upper bounds on TDS for power plants PW1 and PW2.



(a) PW1



(b) PW2

Figure 4-8 Reclaimed Water Supplied for Power Plants PW1 and PW2

The example application evidence indicates the applicability of the model for operation of a WDS under limited electrical power availability. The demands are satisfied during sufficient electrical power availability while the demand satisfaction is curtailed during limited electrical power availability.

4.10. Conclusions

The model presented in this paper is a novel approach for optimization of real-time WDS operations under normal as well as limited power conditions. In addition to minimizing the difference between required and satisfied demands, the model also achieves compliance of the water quality requirements for total dissolved solids (TDS) at the power plants. Current research on the model is the interface with the PDS as illustrated in Figure 4-1.

The model presented in this paper can be used to evaluate the resilience of a WDS to power interruptions and failures. Resiliency of the WDS operations is the extent to which the WDS can recover from failures or disturbances in achieving its hydraulic performance requirements. Future research will include interfacing the assessment procedure developed by Aydin et al. (2014a) to determine system resilience with the model presented in this paper under considerations for both limited water and/or power availability. The interactive nature of the optimization-simulation methodology presented in this study with a resilience computation methodology will provide the WDS utility with a real-time performance analysis and decision-making tool.

Emergency conditions such as drought lead to chronic water shortages, thereby reducing the capability of the WDS to supply water. Further research is required to assess the effects of chronic water shortages on electrical power generation. The model presented in this paper can be further expanded for application to projected water shortage scenarios.

5. OPTIMIZATION-SIMULATION MODEL FOR REAL-TIME PUMP AND VALVE OPERATION OF WATER DISTRIBUTION SYSTEMS UNDER CRITICAL CONDITIONS

5.1. Summary

A new methodology is developed for the real-time operation of water distribution systems (WDS) under critical conditions of limited electrical energy and/or water availability due to emergencies such as extreme drought conditions, electric grid failure, and other severe conditions including natural and manmade disasters. The methodology is tested for three different scenarios of limited electrical energy availability for an example WDS, but it is also applicable for conditions of limited water availability. The basic objective of optimizing the operations of WDS under critical conditions is to minimize the difference between requested demands and satisfied demands while meeting pressure requirements of the system. The approach adopted here is to interface an optimization procedure (genetic algorithm) with a simulator (EPANET) in the framework of an optimal control problem to determine the real-time optimal operation (pump and valve operations) of a water distribution system. Interfacing of the simulator and the genetic algorithm has been accomplished within the framework of MATLAB.

5.2. Background: Electrical Power System and Water Distribution System Interdependencies

The interdependencies of a WDS over the PDS depend upon several factors related to the layout and components of the system. Major energy consumption involved in pumping of water from the source to treatment facility and finally to the consumer.

Rinaldi et al. (2001) present a detailed study on critical infrastructure interdependencies highlighting the ripple effect caused by disruption of power supply on other critical infrastructure systems including WDS. Zachariadis and Poullikkas (2012) present an economic analysis of disruptions to water distribution due to power outages with a case study from Cyprus. Power outages can cause widespread disruption of water supply and hence are a great concern for WDS operations. The interdependencies between power distribution systems (PDS) and water distribution systems (WDS) have been recognized as the water-energy nexus.

The interaction between these two supervisory control and data acquisition (SCADA) systems is to exchange real time data, through software defined networking middleware architecture (see Figure 5-1). The overlay enables a reliable and efficient data exchange between the two otherwise isolated PDS and WDS systems. This is the ultimate measure of how the two systems behave when subjected to various disturbances in either system as well as under conditions of long-term water shortages. Control actions undertaken in both networks represent an improvement over the current practices.

This paper addresses the development of an optimization/simulation model for the real-time operation of water distribution systems that would operate under the critical conditions of limited electrical energy supply and/or limited water availability. The novelty of this model is that it can be used to determine optimal real-time pump and valve operations under critical conditions.

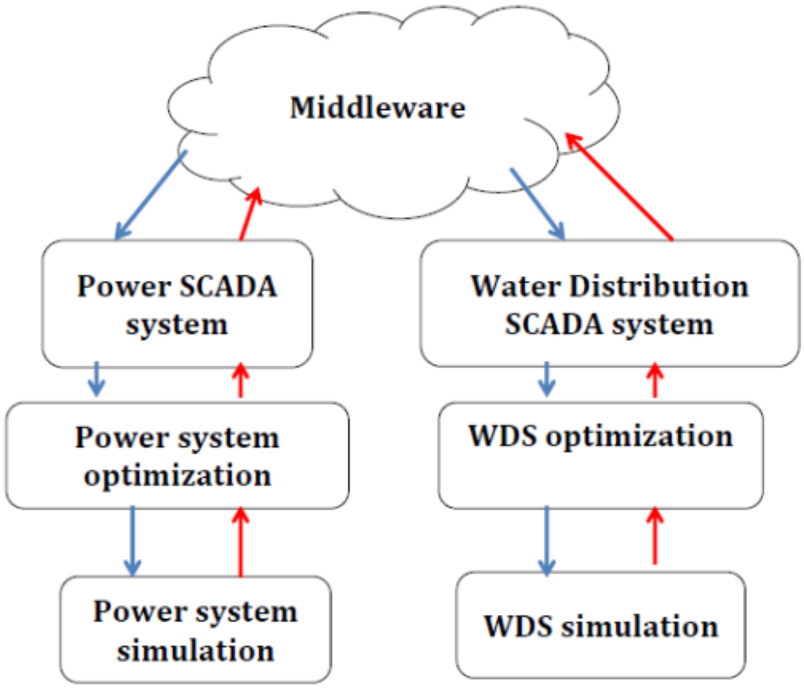


Figure 5-1 Real-time Operation Model for Optimal Operation of Power – Water Systems

5.3. Previous Optimization Models for Pump Operation

Goldman et al. (2002) provide a detailed list of optimization models developed for pump operations. Models optimizing pump operations in water distribution systems (WDS) developed after 2002 include works by: Rao and Salomons (2007); Sakarya and Mays (2000); Salomons et al. (2007); Shamir and Salomons (2008); Ramos and Ramos (2009); Cohen et al. (2009); Costa et al. (2010); Kurek and Ostfield (2013); Goldman et al. (2002); Hashemi et al. (2014); Ghaddar et al. (2014) and Khatavkar and Mays (2017c).

There have been a few optimization/simulation models for pump operation reported in the literature that have interfaced optimization models with EPANET

(Rossman, 2000). EPANET (developed by the U.S. Environmental Protection Agency) is the commonly used open source software for hydraulic and quality simulations

Sakarya and Mays (2000) developed a model using nonlinear programming interfaced with EPANET for both water quality and hydraulic performance requirements. Goldman and Mays (2005) developed a methodology that linked the simulated annealing with the EPANET model to find optimal pump operation for WDS while meeting both water quality and hydraulic performance requirements. Ozger and Mays (2005) interfaced a simulated annealing model with EPANET for determining the optimal location of isolation valves in water distribution systems for security purposes.

Kurek and Ostfeld (2013) linked EPANET with a multi-objective methodology using a strength pareto evolutionary algorithm to demonstrate the trade-offs between pumping cost, water quality and tank sizing of WDS. Costa, et al (2010) developed a branch and bound algorithm interfaced with EPANET for optimal pump operation of WDS.

5.4. Previous Real-time Optimal Pump Operation Models

Several attempts at developing models for the real-time operation of water distribution systems have been reported in the literature. Most of these efforts have simplified the hydraulics such as models by Rao and Alvarruiz (2007), Rao and Salomons (2007), Shamir and Salomons (2008), and others with more detail below. Broad et al. (2005) and Salomons et al. (2007) used a surrogate hydraulic model (or meta-model) "trained" to approximate the nonlinear hydraulic equations using a machine learning approach which is a support vector machine that replaces EPANET. Cheng, et al. (2014) presented an approach for real-time operation of a WDS using a state estimation method with measured data via a SCADA and a steady-state WDS simulation model. To develop real-time pump scheduling tools, Pasha and Lansey (2014) presented two approaches (a linear programming model and the shuffled frog leaping algorithm by Eusuff et al. 2006) to find approximate solutions which can be used as warm solutions in evolutionary algorithms to reduce computation time.

Other investigators have proposed the use of an artificial neural network. These include Jamieson et al. (2007); Broad, et al. (2010); and Odan et al. (2014) who substituted an artificial neural network for a hydraulic simulation model to speed up the process. Odan et al. (2014) used an adaptive ANN training (Islam et al. 2009) for the simulation of the water distribution system and used a multi-method optimization algorithm as an alternative to a genetic algorithm (GA). The multi algorithm genetically adaptive method, proposed by Vrugt and Robinson (2007) was chosen as the optimization method to solve the pump scheduling problem for real-time operation. Kang (2014) developed a real-time optimal control model for pump operation in WDS to minimize operation costs which interfaced a GA with the EPANET model. As pointed out by Kang (2014) "the contemporaneous portion of the current GA decisions can be used as initial starting conditions for the next GA runs."

Boulos, et al. (2014) developed a real-time model referred to as smart water network decision support system. As stated by the authors, "The mart water network decision support system runs in near real time by reading SCADA measurements as they become available, updating the network model boundary conditions and operational statuses, pausing execution and generating the corresponding network analysis results, and then waiting for the new SCADA measurements to reload the network model and rerun the network simulation." The authors applied this real-time network modeling system to the Las Vegas Valley Water District in Las Vegas, Nevada to reduce water age and enhance water quality and to decrease the operational cost of the network through improved daily operating plans and pump control rules.

Odan et al. (2015) developed a multi-objective optimization model for real-time operations of water supply pumps using an interface between a water demand forecasting model, EPANET and an optimization model for minimization of energy utilization. Candelieri et al. (2018) presented a Bayesian optimization approach for pump operations for WDS using probabilistic functions and EPANET for simulation of the results. Mala – Jetmarova et al. (2018) provide a detailed literature review for recent optimization modes for WDS operations and design.

Limited research related to optimization of WDS controls under critical conditions of limited power availability is available in the literature. Li et al. (2018) presented an optimization model for operations of water-energy nexus systems. The model presented by Li et al. (2018) considered optimization of pump schedules during power outages. Khatavkar and Mays (2018) proposed an optimization-simulation model for real-time control of water distribution pumps under the conditions of limited power availability. No research is reported in the literature that provides a comprehensive optimization-simulation model for both pump and valve operations under conditions of limited power availability and/or water availability. The overall effort of the research reported in this paper is to expand the Khatavkar and Mays (2018) optimization-simulation model for the pump and valve controls of WDS that will communicate with an

PDS optimization – simulation model through the middleware as illustrated in Figure 5-1. This paper presents a new optimization-simulation model for the real-time operation of water distribution systems that would operate under the critical conditions of limited electrical energy supply and/or limited water availability. The novelty of this model is that it can be used to determine optimal real-time pump and valve operations for a WDS.

5.5. Real-time Operation Framework

The basic objective of optimizing the operations of a water distribution system is an attempt to satisfy requested demand in a WDS while satisfying pressure requirements. The objective statement considers a particular requested demand $D_{req,z,t}$ Qb_z, where $D_{req,z,t}$ is the demand coefficient for service area z at a time t and Qb_z is the base demand for service area z. The average of demand coefficients over a 24-hr time-period must be 1.0, since a 24-hr average daily water demand is used as the base demand for WDS simulation. Forecasting water demands for real-time operations of WDS can be achieved through use of statistical forecasting methodologies or average water demands. This study uses average water demands for the real-time WDS operations. The following describes the overall concept of the real-time operation for pump and valve controls.

1. At real-time t receive electrical energy input from electrical system and water availability from water utilities.
2. Receive latest data (status of pumps, tank levels, status of valves, and flows in and out of the system from the SCADA (Supervisory Control and Data Acquisition) system.

3. Update the EPANET water distribution system (WDS) input using data from the SCADA system.
4. WDS optimization model is run to determine the actual demand pattern and pump operation that can be met with the limited electrical energy input. During the optimization/simulation process the EPANET model is used repeatedly within the GA optimization to determine the status of the network over the next 24 hours. The optimization model searches over the decision variables which are the pump operations and/or valve operations to determine the optimal demand pattern that minimizes the difference between the requested demands and satisfied demands. The simulator determines the values of the state variables (nodal pressure heads, pipe flows, and tank levels) for each set of control variables determined in the optimizer.
5. Implement the optimal pump schedule over the next one hour only which is accomplished through the SCADA system.
6. Repeat steps 1 through 5 continuously during the emergency event each time incrementing the time $t = t + \Delta t$ in which case $\Delta t = 1$ hour.

5.6. WDS Optimization Model

The actual demand that can be satisfied in service area z during time-period t is $\mathrm{D_{sat,z,t}Qb_z}$. The objective of WDS operation is to minimize the difference between demands requested and demands satisfied; however, if energy and/or water availability are not sufficient, the requested demand is not satisfied then $\mathrm{D_{sat,z,t}} \leq \mathrm{D_{req,z,t}}$.

The following is the problem statement for the WDS optimization model.

5.6.1. Objective Function

The objective function for the WDS optimization model can be expressed as

$$\text{Minimize } Z = \sum_{t}^{t=T} \sum_{z}^{z=N} [(D_{req,\, z,t} - D_{sat,z,t}) Qb_z]^n P_{z,t} \qquad (5\text{-}1)$$

where $P_{z,t}$ is a penalty for not meeting the requested demands in service area z at time t. The objective function is subjected to several constraints, those solved by the simulator EPANET and bound constraints incorporated into the objective functions as penalty functions.

5.6.2. Model Constraints Solved by EPANET

The distribution of flow throughout the network must satisfy the conservation of mass and the conservation of energy which are defined as the hydraulic constraints. The hydraulic constraints are solved implicit to the optimization model using EPANET (Rossman, 2000) and are documented in detail by Khatavkar and Mays (2018).

5.6.3. Bound Constraints Incorporated into Reduced Problem

Additional inequality constraints, which are the bound constraints, include bounds on nodal pressure heads, pump operation times, number of times pumps can be turned off and on (switches), bounds on demand satisfaction multiplier, bounds on valve operation, and bounds on power availability. These constraints cannot be solved using the simulator, EPANET. They are incorporated into the reduced problem which is solved using the GA.

Nodal pressure head bounds are expressed as

$$\underline{H}_{kt} \leq H_{kt} \leq \overline{H}_{kt} \qquad \forall\ k = 1, ..., K \text{ and } t = 1, ..., T \qquad (5\text{-}2)$$

in which $\underline{H}_{kt}$ and $\overline{H}_{kt}$ are the lower and upper bounds, respectively on pressure head at node k at time t, H_{kt}. There are no universally accepted values for the lower and upper bound values.

The lower and upper bounds on pump operation time are expressed as,

$$\Delta t_{min,p} \leq \sum_t X_{p,t} \leq \Delta t_{max,p} \qquad \forall\ p = 1, ..., P \text{ and } t = 1, ..., T \tag{5-3}$$

where $X_{p,t}$ is a binary variable (0 or 1) depicting the pump operation for pump (p) at time t, and $\Delta t_{min,p}$ and $\Delta t_{max,p}$ are the lower and upper bounds on total number of hours the pump p is switched on respectively. Δt_{min} can be zero to simulate a closed pump and Δt_{max} is equal to the maximum length of time period pump p could be operated during a particular simulation period. Switch constraints define the minimum and maximum number of times a pump is turned on referred to as the number of switches expressed as

$$Sw_{min(p)} \leq \sum_t \max\left(0, (X_{p,t} - X_{p,t-1})\right) \leq Sw_{max(p)}$$

$$\forall\ p = 1, ..., P \text{ and } t = 1, ..., T \tag{5-4}$$

where $Sw_{min(p)}$ and $Sw_{max(p)}$ are the minimum and maximum number of switches allowed for the pump p. Bounds for demand satisfaction multiplier, $D_{sat,z,t}$ are

$$a\, D_{req,\,z,t} \leq D_{sat,z,t} \leq D_{req,\,z,t} \qquad \forall\ z = 1, ..., Z \text{ and } t = 1, ..., T \tag{5-5}$$

where $a \leq 1$.

Flow control valves (FCV) play an important role in maintaining required pressures at the nodes and flows in the pipes in a WDS. Flow control valves are used to control the flow to node or a service area in the WDS (Jowitt and Xu, 1990). For control of WDS under conditions of limited electrical energy input, valve operations are introduced in the model presented following equation with a binary variable ($X_{v\,(iz,t)}$) for valve controls. The constraint for valve operations for a valve between two consecutive nodes i and service area z is as follows

$$X_{v\,(iz,t)}\, Q_{iz,t} \geq D_{sat,z,t}\, Qb_z \qquad \forall\, z = i = 1,\ldots,Z,\ t = 1, \ldots, T \tag{5-6}$$

where $X_{v\,(iz,t)}$ is a binary variable (0 for valve closed, 1 for valve open) showing operations of a valve between node i and service area z and $Q_{iz,t}$ is the flow from node i to service area z

5.6.4. Consideration of Power Availability During Limited Energey Availability

The above optimization model defined by equations 5.1, 5.2, 5.4, 5.5 and 5.6 is expanded to determine the optimal pump operation to maximize the amount of demands that can be satisfied. The relation of available pump discharges to power available from the power distribution system (given by Khatavkar and Mays, 2017 c) is expressed as

$$X_{p,t} \frac{Q_{p,t}\, H_{p,t}}{3956\eta_p} \leq P_{Avail,t} \tag{5-7}$$

where $P_{Avail,t}$ is the total power available for the water distribution system for time period t; $X_{p,t}$ is a binary variable (0 or 1) depicting the pump operation for pump p at time t as described above; and η_p is the efficiency of pump p.

The optimization model described above must be reformulated as an unconstrained optimization model for solution using a GA. Constraints on the conservation of mass and energy and storage tank continuity are solved implicitly to the optimization using the EPANET model and the remaining bound constraints (equations 5.2, 5.3, 5.4, 5.5 and 5.6) are incorporated into the objective function as penalties resulting in the reduced problem which is an unconstrained optimization problem solved by the GA. The optimizer (in this model a GA) searches over the control variable (pump operation times) using the values of the state variables (nodal heads, pipe flows, and tank levels determined in the simulator. In other words, the optimizer passes pump and valve operations to the simulator which determines the state variables (nodal heads, pipe flows, and tank levels). This process continues until an optimal or near optimal solution is reached.

5.7. Reduced Optimization Problem Solved by Genetic Algorithm

The reduced problem is an unconstrained optimization problem developed by incorporating the above bound constraints (equations 5.2, 5.3, 5.4, 5.5, 5.6, and 5.7) into the objective function (1) as penalty functions. The reduced optimization problem, described by equation 5.8 below is solved using a GA:

$$\text{Minimize } Z_{\text{reduced}} = W_{1,t} \sum_{t}^{t=T} \sum_{z}^{z=Z} \left[\left(D_{\text{req, z,t}} - D_{\text{sat,z,t}} \right) Qb_z \right]^n P_{z,t}$$

$$+ W_{2,t} \sum_{t}^{t=T} \sum_{k}^{k=N} \left(w_1 \left(\max\left(0, \left(H_{k,t} - \overline{H_n}\right)\right)\right)^2 + w_2 \left(\min\left(0, \left(H_{k,t} - \underline{H_k}\right)\right)\right)^2 + \right.$$

$$\left. w_3 \left(\min\left(0, \left(H_{k,t} - 0\right)\right)\right)^2 \right) + W_{3,t} \left[\sum_{p}^{p=Pu} \left(\max\left(0, \left(\Delta t_{\min,p} - \sum_{t}^{t=T} X_{p,t} \right)\right)\right)^2 + \right.$$

$$\left. \sum_{p}^{p=Pu} \left(\max\left(0, \left(\sum_{t}^{t=T} X_{p,t} - \Delta t_{\max,p} \right)\right)\right)^2 \right] + W_{4,t} \left[\sum_{p}^{p=Pu} \left(\max\left(0, \left(Sw_{\min(p)} - \right.\right.\right.\right.$$

$$\left.\left.\left. \sum_t \max\left(0, \left(X_{p,t} - X_{p,t-1}\right)\right)\right)\right)\right)^2 + \sum_{p}^{p=Pu} \left(\max\left(0, \left(\sum_t \max\left(0, \left(X_{p,t} - X_{p,t-1}\right)\right) - \right.\right.\right.$$

$$\left.\left.\left.\left. Sw_{\max(p)} \right)\right)\right)^2 \right] + W_{5,t} \sum_{t}^{t=T} \sum_{p}^{p=Pu} \left(\max\left(0, \left(X_{p,t} \frac{Q_{p,t} H_{p,t}}{3956 \eta_p} - P_{\text{Avail}(p,t)} \right)\right)\right)^2 +$$

$$W_{6,t} \sum_{z}^{z=Z} \sum_{i}^{i=Z-1} \left(\min\left(0, \left(X_{v\,(iz,t)} Q_{iz,t} - D_{\text{sat,z,t}} Qb_z \right)\right)\right)^2 \qquad (5\text{-}8)$$

where $W_{m,t}$ is the penalty weight associated with term m for time t; n is the index associated with the objective term: w is the weight associated with a pressure constraint;

Z is the total number of demand patterns in the system; Pu is the total number of pipes in the system; N is the total number of tanks plus the number of nodes monitored for pressure; and P_z is the priority associated with demand pattern z.

The penalty weights $W_{1,t} – W_{6,t}$ are associated with demand satisfaction, pressure constraints, power constraints, total power utilization, switch constraints and valve constraints, respectively. The penalty weights are determined through a sensitivity analysis for each application of the optimization model. The sensitivity analyses are performed for all terms involved in the reduced optimization model. The value of weights to be used for a particular term depend upon the relative importance of the penalty term and the magnitude of the numeric value of the penalty term. Parameters considered for the sensitivity analyses are the value of the objective function, number of pressure bound violations, number of power bound violations and the number of switch constraint violations. The optimization model is tested for a range of values for each of the penalty weights and a combination of penalty weights yielding the minimum objective function value and least amount of bound violations for pressures, power and water quality is chosen for the application.

EPANET is a demand-driven simulation model that always meets the demands irrespective of pressures at nodes in the WDS. The solutions may have negative pressures at certain nodes for operations where enough water is not pumped to meet the water demands. For every iteration, the GA provides a new set of decision variables (including satisfied demand multipliers and pump and valve controls) and an EPANET simulation is performed based on the decision variables. A negative pressure penalty function with a large penalty weight is used in the reduced optimization model to exclude EPANET solutions with negative pressures. The final solution from the optimization-simulation can never include an operation schedule that results in negative pressures.

5.8. Optimization-Simulation Model

The overall model interface between the WDS model and the EPANET simulator is shown in Figure 5-2 is accomplished using MATLAB. Within MATLAB communication with EPANET is performed using the open-source EPANET-MATLAB toolkit (Eliades, 2016). The interface facilitates use of all the functionalities in the EPANET code within the MALTAB environment by passing the various commands to and from the MATLAB mathematical language to the EPANET simulator. This toolkit was used in conjunction with GA in MATLAB to accomplish an overall optimization/ simulation methodology for a WDS. The methodology works through an interface facilitating data exchange between three systems including the optimization model to solve (reduced problem), WDS Supervisory Control and Data Acquisition (SCADA) system and EPANET as illustrated in Figure 5-2.

Figure 5-3 is a schematic of the overall simulation/optimization procedure being implemented for every unit time period for the entire simulation period. Starting at the first-time period, the optimization/simulation model in MATLAB receives inputs from the power optimization/simulation model including observed and predicted power availability schedule for each time period and data from WDS SCADA system including pressure heads, discharges and tank levels as well as pump status at various nodes and links in the system. The optimization/simulation model for WDS is then run with the updated information to obtain the pump controls, valve controls and demand satisfaction multipliers for the next time. This process continues until the GA stopping conditions are met. The solution of the GA is then sent to the power optimization/simulation model and the WDS SCADA system as inputs for the next time until the last time period for simulation is reached.

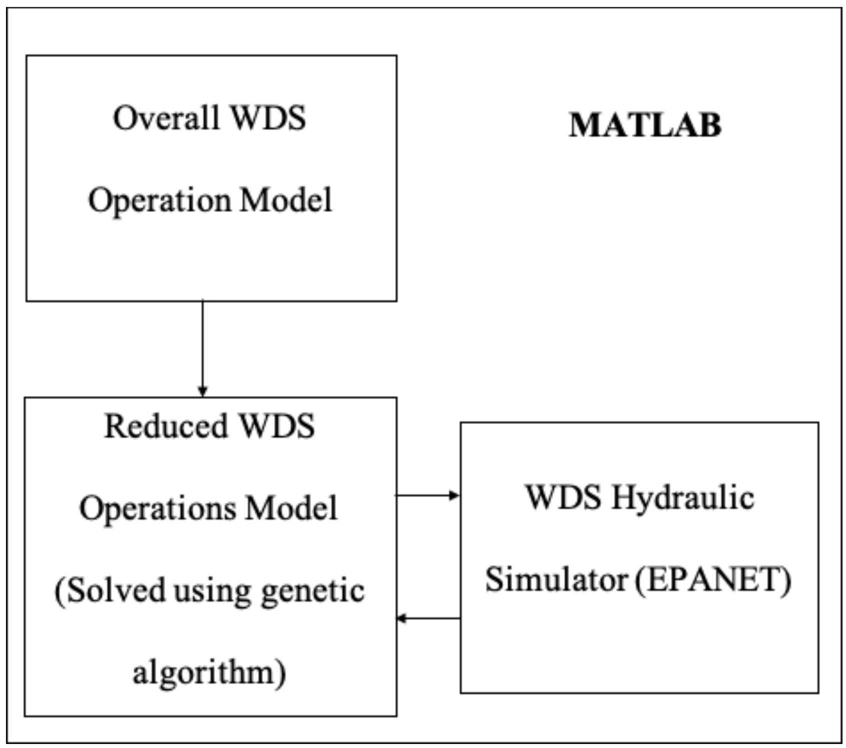


Figure 5-2 Water Distribution Simulation/Optimization Model in MATLAB

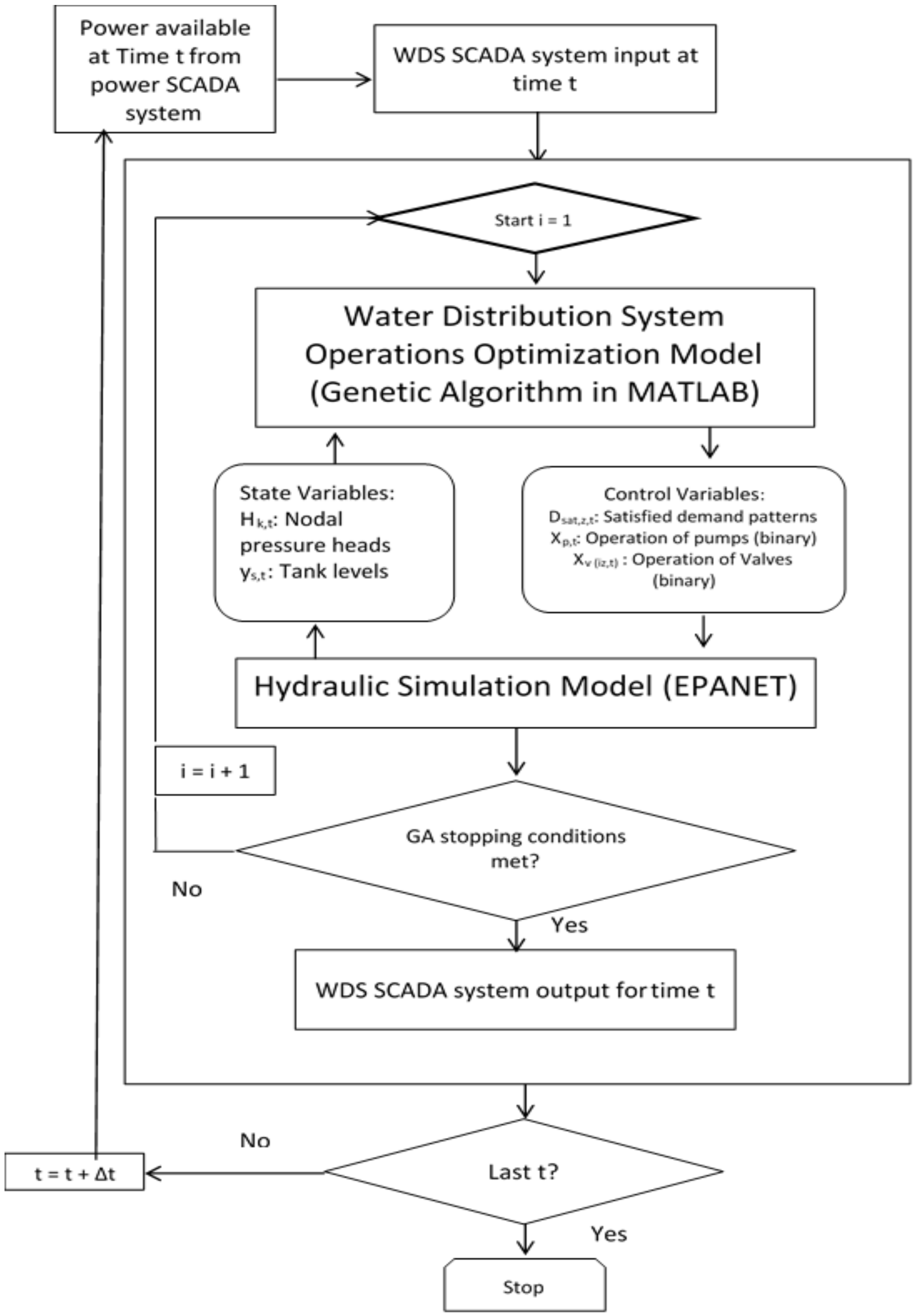


Figure 5-3 Schematic of the Overall Simulation/Optimization Procedure

5.9. Example Application

The WDS network shown in Figure 5-4 is used to illustrate the applicability of the optimization - simulation model. The WDS consists of 92 nodes, 117 pipes and 5 service areas. Service area 1 was modeled for the purposes of this application. The remaining service areas are comparatively smaller regions modeled as single nodes with individual demand patterns. Table 5-1 gives the demand types and total base demand (Qb_z) for the service areas in WDS. The system includes three tanks, two sources of water, and two pumps. This network has been used over the years as an example by several investigators including Vasconcelos et al. (1996), Sakarya and Mays (2000), Goldman and Mays (2005), Bagirov et al (2013) and others.

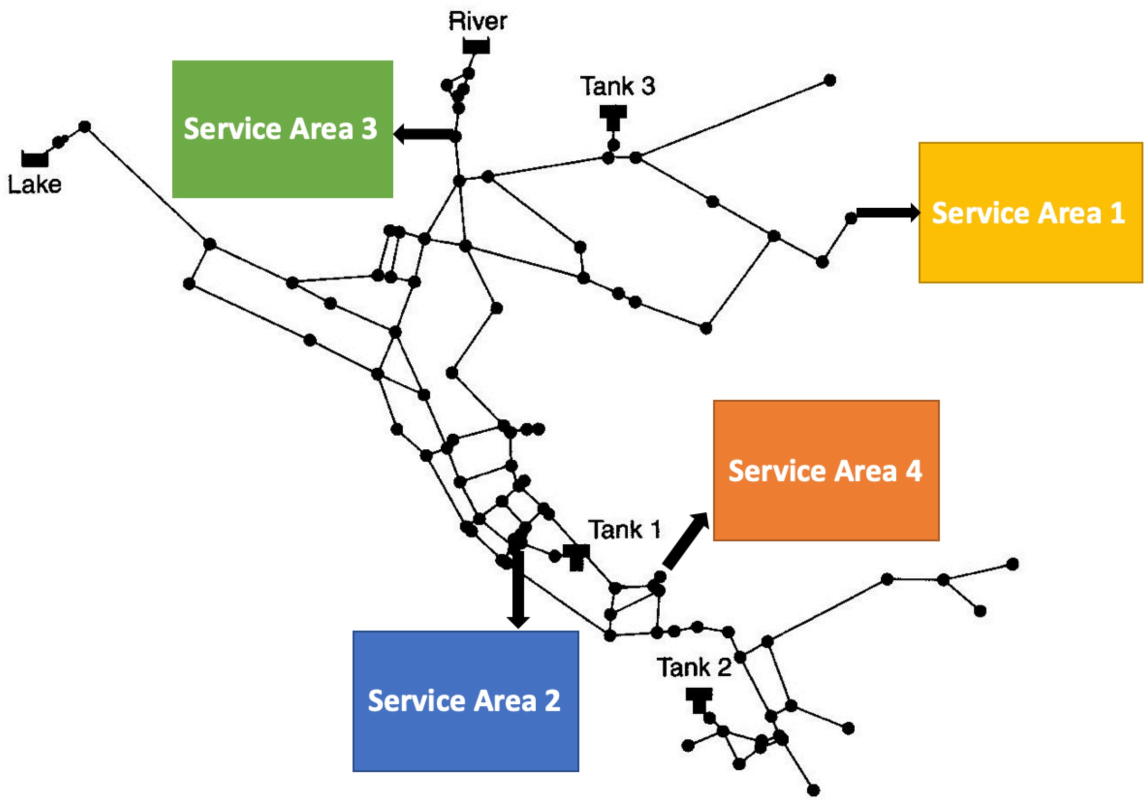


Figure 5-4 Example Water Distribution System Network Showing the Connection to the Four Other Service Areas.

Table 5-1 Details of Demands for WDS

Service area (z)	Type of demand	Total base demand Qb_z (in gpm)
1	Residential/ Irrigation	2970
2	Commercial	1859
3	Commercial	620
4	Residential	1856
5	Irrigation	4643

Inputs for the GA include the number of function calls (iterations), weights for different penalty terms, and the fitness function for the solution. Results of several sensitivity analyses show that a total of 1100 function calls provides a solution with minimum pressure and power constraint violations. The results for the three scenarios considered required a total of 982, 952 and 1086 function calls (number of times EPANET model is solved) respectively. The time taken for obtaining the results for the three scenarios were 13.09, 12.69 and 14.48 minutes respectively. The time taken for obtaining the GA results was much less than the operations unit time of one hour. Therefore, the GA can be effectively used for real-time WDS operations. Larger WDS networks would need skeletonizing for application of the methodology. Pressure bounds were applied for all the nodes in the system except for the nodes representing tanks in the WDS. The upper and lower bounds for tank heads (levels) used were 151 ft and 149 ft respectively. The pressure heads at service area nodes were bound between 100 psi and 40 psi respectively. No bounds were used for pump operations in this application.

An interface between the WDS optimization-simulation model and a PDS optimization-simulation model is being developed as a part of the ongoing research for NSF Project 029013-0010, CRISP Type 2—Resilient cyber-enabled electric energy and water infrastructures modeling and control under extreme mega droughts for combined operations under emergency conditions of limited power and/or water availability. This combined model enables simulations of long-term emergency conditions such as droughts or other natural and manmade disasters as a tool for contingency planning. The power availability at different pumping stations in the WDS would be communicated by the PDS optimization-simulation model at real-time.

Three scenarios were considered in the example application: (1) observed pump schedules and demands without optimization from the simulation model, (2) optimization - simulation model application for limited power availability for pump controls and (3) optimization - simulation model application for limited power availability for pump controls and valve controls. In the second scenario, only pump controls were considered for optimization using the model for pump controls only while the third scenario considered optimization of both pump controls as well as valve controls using the model for pump and valve controls. The values used for the weights $W_{1,t}$ through $W_{6,t}$ for scenarios 2 and 3 below are 10^8, 10^5, 10^{50}, 10^0, 10^5 and 100 respectively. A lower penalty weight was used for the upper bound on pump switches, since during short-term emergency conditions pump switches are not a major concern. For long-term operations of the WDS pump switches are an important consideration. The total number of pump switches observed for the three scenarios were 10, 12 and 11 respectively.

5.9.1. Scenario 1- Observed Pump Schedules and Demands Without Optimization

Through application of the EPANET model the power requirements for the pump observed operations were determined. The results of this simulation provide a basis for comparison of the results for other two scenarios. The observed power requirements for meeting the requested demands are presented in Figure 5-5. The requested and satisfied demand patterns (multipliers) are presented in Figure 5-6 (a) - (e) for service areas 1 to 5, respectively. These figures illustrate that all the demands were satisfied in this case, since the simulation was performed assuming a 100 % demand satisfaction.

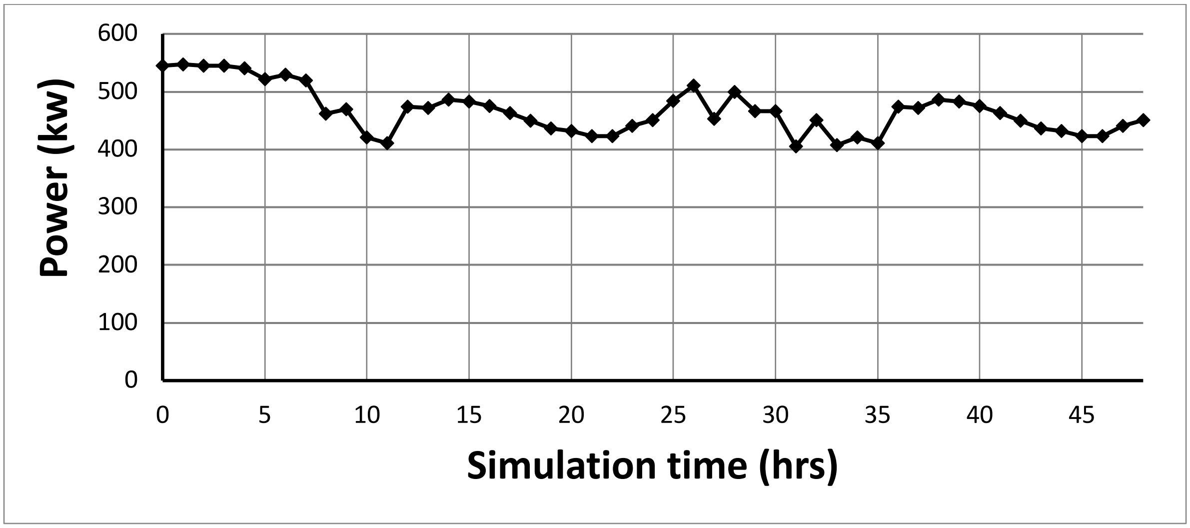


Figure 5-5 Required Power for Observed Pump Operations and Demands for Scenario 1

5.9.2. Scenario 2 – Optimization/Simulation Model Application for Pump Controls Under Limited Power Availability

In this scenario, a limited amount of power availability for a few hours of the day was considered. The optimization/simulation model (developed for pump controls with a maximum demand satisfaction objective) was applied. The power availability as a function of time as illustrated in Figure 5-7 compared with the power consumption. The power disruptions considered in this study represent worst case scenarios since the limited power availability (shown in Figure 5-7) is at peak water demand hours. For all times, the power consumption is within the limits of the power available, which shows that there were no power constraint violations in the application of the optimization/simulation model. Figure 5-8 (a) – (e) illustrate the demand requirement

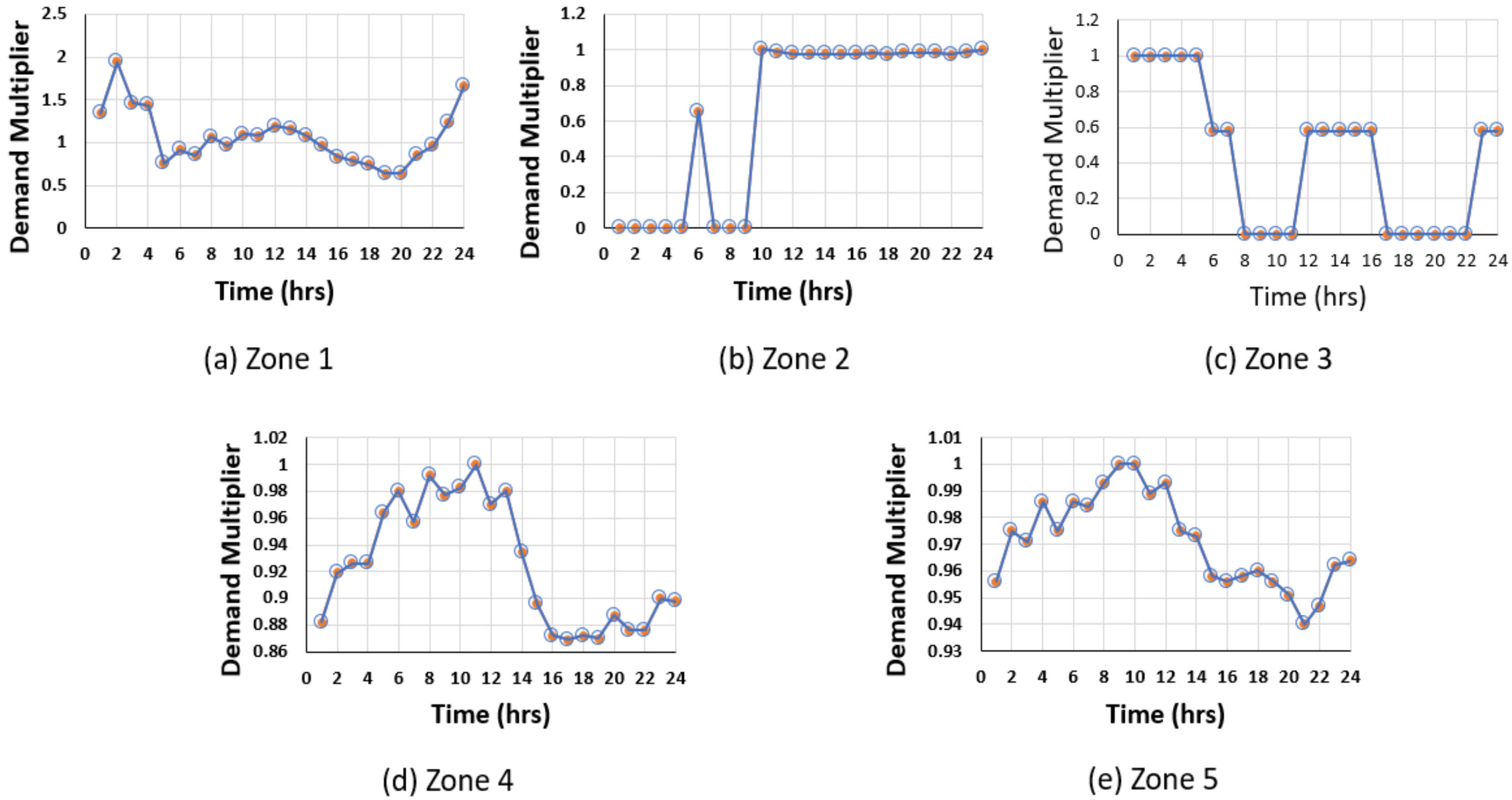


Figure 5-6 Demand Requirement for Observed Conditions

and demand satisfaction (multipliers) patterns for service areas 1 – 5 in the network. The results obtained for this application had an average demand satisfaction of about 78%.

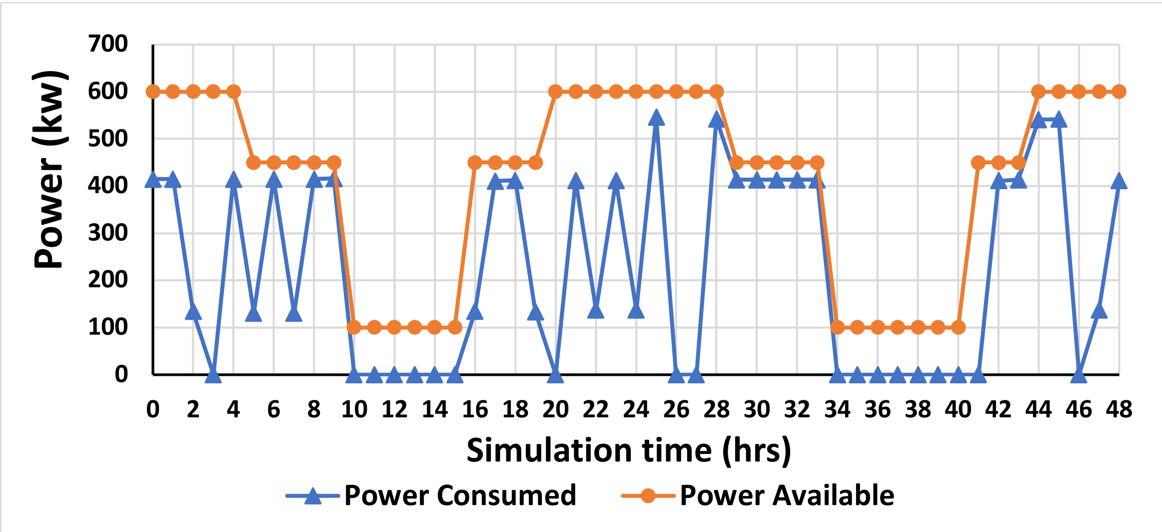


Figure 5-7 Power Availability and Power Consumption for Optimized Pump Schedules and Demand Satisfaction for Scenario 2

Requested demands for the service areas in the system are given in Figure 5-6 (a – e). Since scenario 1 provides the observed pump schedules and demand patterns, the requested demands are assumed to be met in this case and no power bounds were imposed. Figure 5-7 gives the power availability and consumption trends for the system considered in the scenario 2 run performed as a part of this study. There are two limited power availability periods hours 10 – 15 and hours 35 – 40 in this power availability pattern. The results for this scenario show that both the pumps were switched off during these times of limited power availability since the power available was not enough to run either of the pumps. The model attempts to find the most optimum results with maximum possible demand satisfaction. Figure 5-8 (a – e) show the patterns of demands requested and the demands satisfied by the optimized operations of the WDS for scenario 3 for the five service areas in the system respectively.

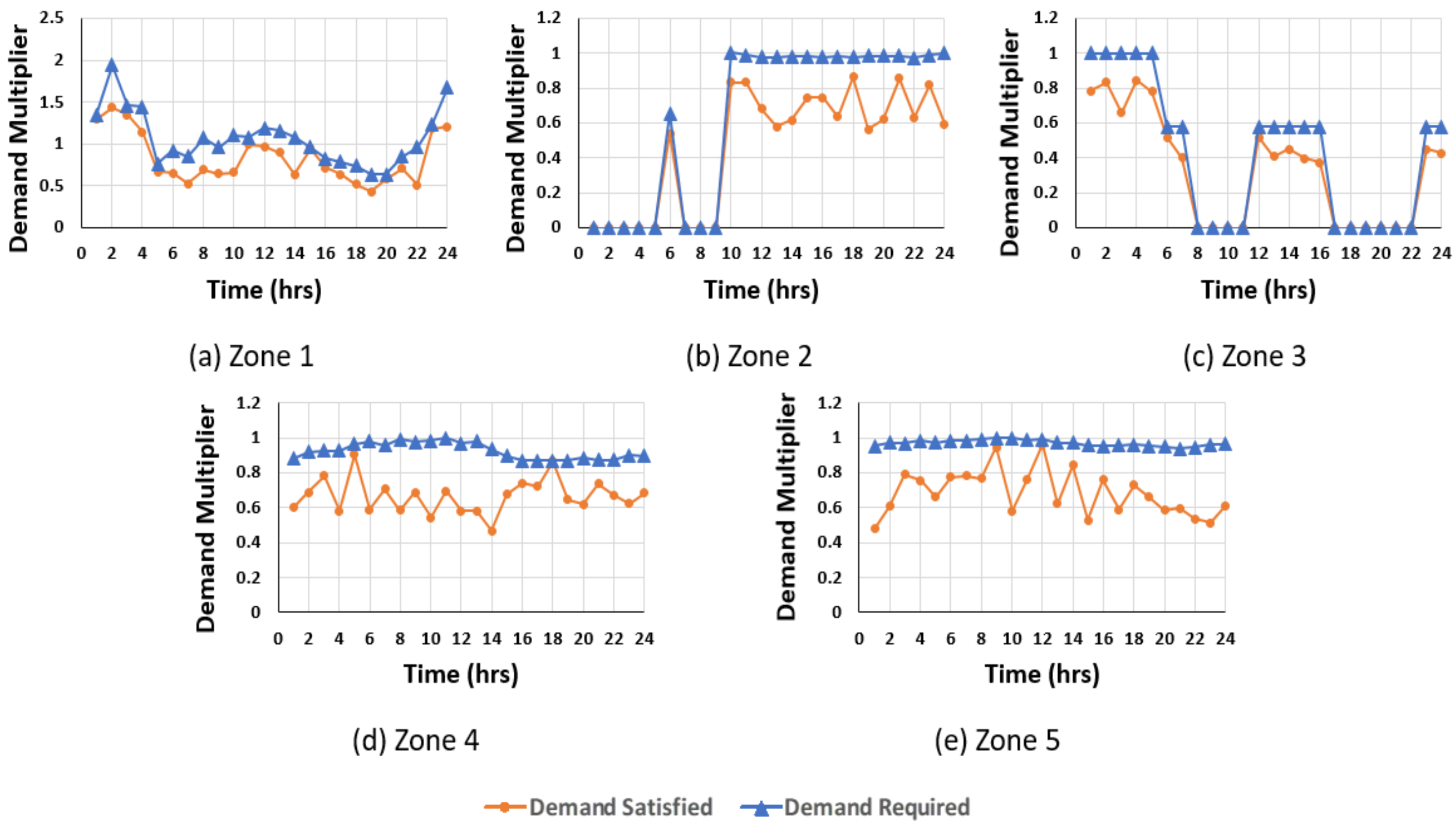


Figure 5-8 Demand Requirement and Demand Satisfaction Multipliers of the Five Service Areas for Scenario 2

These result show that the model attempts to find a solution of the satisfied demand patterns, with a minimum difference between the demands requested and the demands satisfied during various times of the simulation period.

5.9.3. Scenario 3 – Optimization/Simulation Model application for Optimal Pump and Valve Operations Under Limited Power Availability

The model was applied to the limited power availability conditions in scenario 2. The flow control valves (FCV) that were used for control optimization were those at the downstream of the three tanks in the system. These valves control the water flow from the tanks to the WDS. Figure 5-9 gives the power availability and consumption for scenario 3 run of the system considering pump and valve controls. No power availability bound violations were observed for this run. Figure 5-10 (a – e) show the patterns of demands requested and demands satisfied for the system. The demands are observed to be satisfied for most of the simulation period except for the periods of lowest power availability. The average demand satisfaction for scenario 3 was about 87%, which is a considerable increase from the 78% observed in scenario 2. Comparing Figure 5-8 (a – e) with Figure 5-10 (a – e), better demand satisfaction is observed in case of scenario 3 for the entire simulation period as compared to scenario 2 results. This is attributed to the improved controls on account of pump and valve control optimization in scenario 3 application. Figure 5-11 (a – c) show the optimized operations of the three valves in the system.

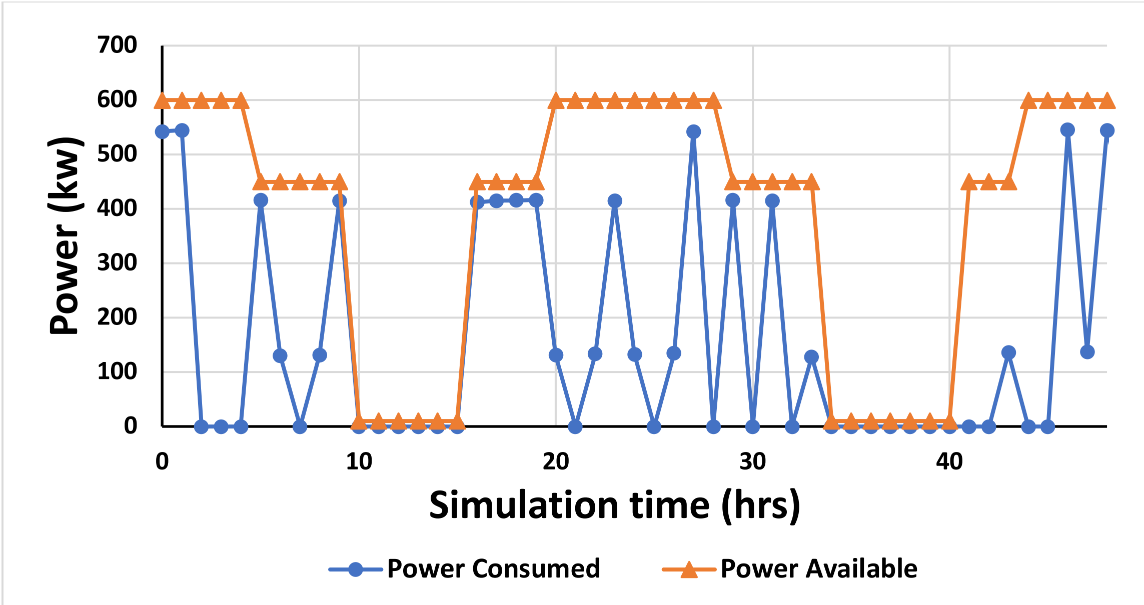


Figure 5-9 Power Availability and Power Consumption for Optimized Pump Schedules and Demand Satisfaction for Scenario 3

5.10. Summary and Conclusions

The algorithm presented is a novel methodology for determination of optimal operations for water distribution systems under normal and emergency conditions of limited power availability and/or water availability. The model employs a methodology for minimizing the difference between the demands requested and the demands satisfied to optimize pump and valve operations of the WDS.

The example application of the model provides evidence of applicability of the model for real-time operations of a WDS under limited power availability. The demands are observed to be satisfied in the period of sufficient power availability while the demand satisfaction is curtailed in case of limited power availability time periods. Improved demand satisfaction was observed with consideration of both pump and valve controls.

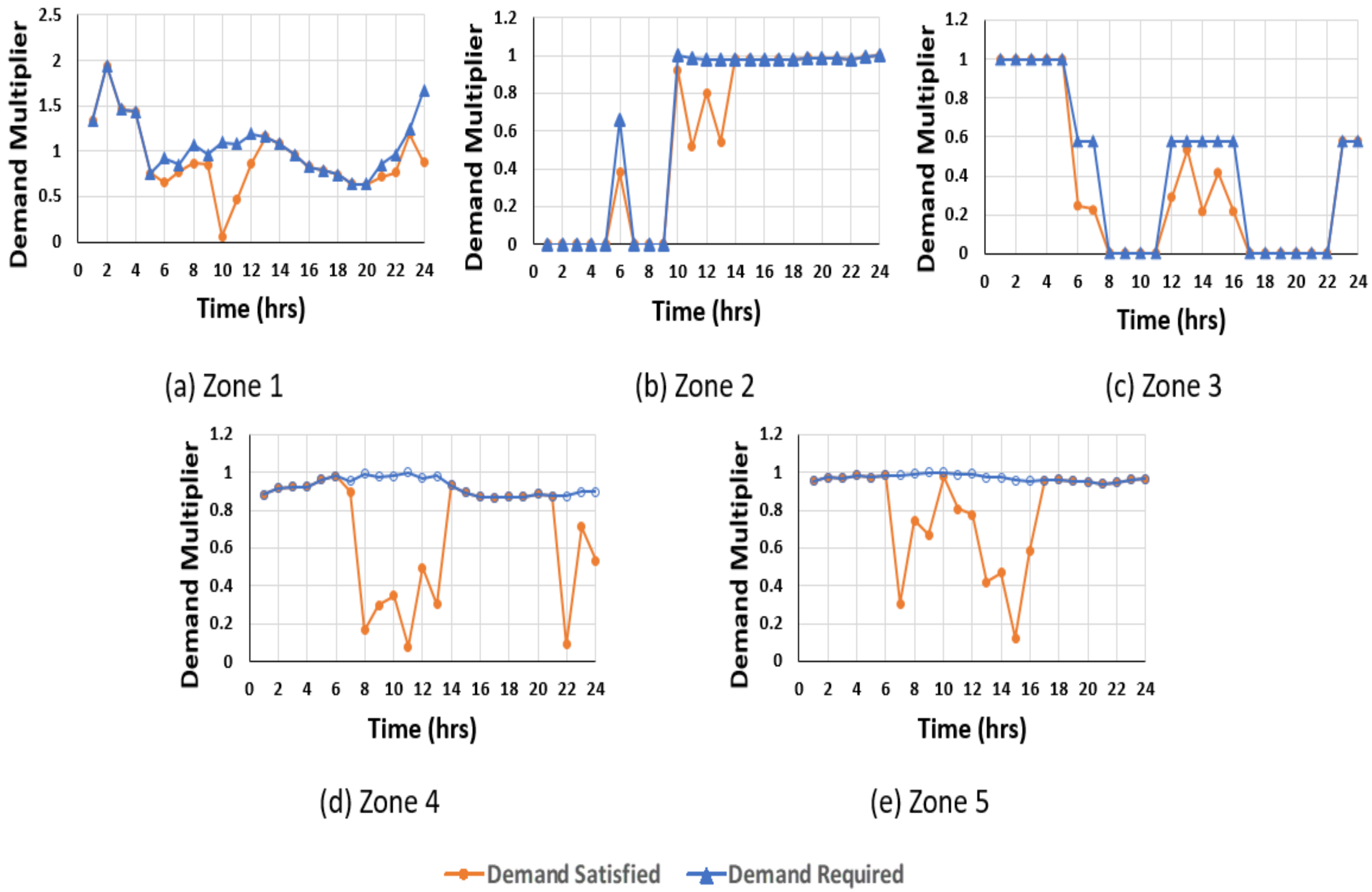


Figure 5-10 Demand Requirement and Demand Satisfaction Multipliers of the Five Service Areas for Scenario 3

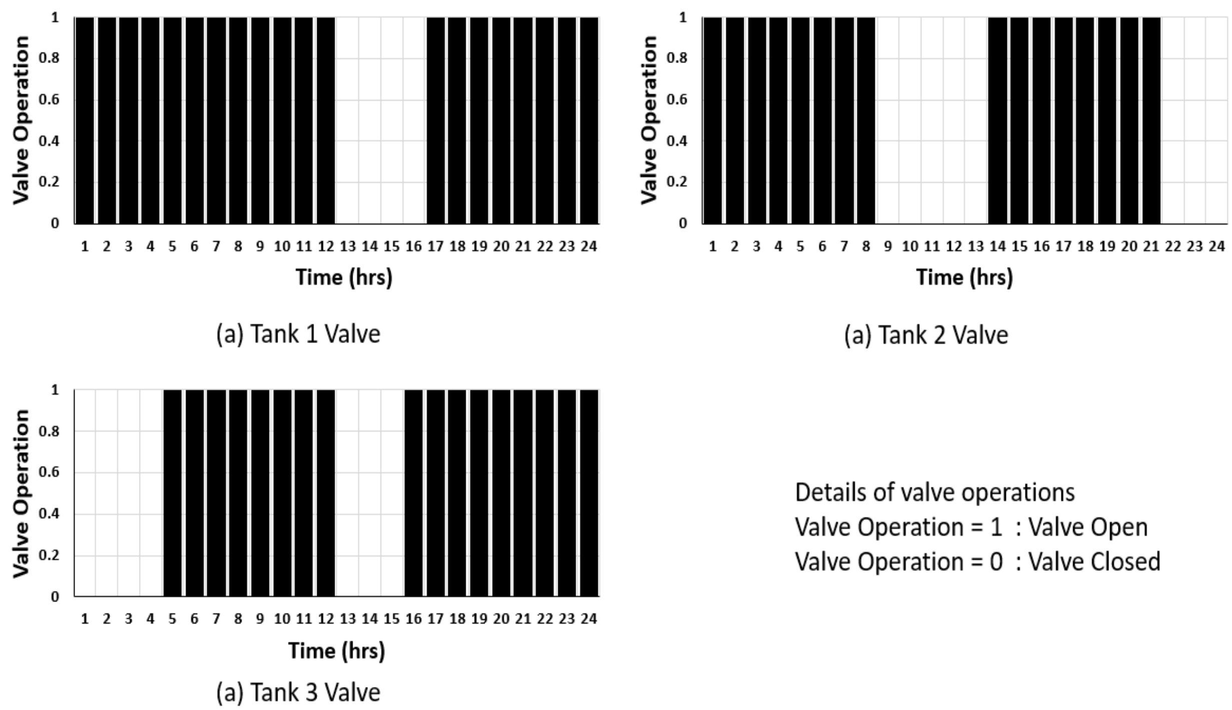


Figure 5-11 Details of valve operations for scenario 3

A few potential limitations of this study are identified. No forecasting model for water demands was used for this study. Another limitation of the model is that random disruptions of the system are not considered. An interface between the WDS optimization-simulation model and a PDS optimization-simulation model is being developed as a part of the ongoing research for NSF Project 029013-0010, CRISP Type 2—Resilient cyber-enabled electric energy and water infrastructures modeling and control under extreme mega droughts for combined operations under emergency conditions of limited power and/or water availability. This combined model would enable simulations of long-term emergency conditions such as droughts or other natural and manmade disasters as a tool for contingency planning. The power availability at different pumping stations in the WDS would be communicated by the PDS optimization-simulation model at real-time. Future work would also involve development of a model for generation of random power disruptions, to be used in conjunction with the optimization-simulation model presented in this study.

The methodology presented in this paper could be used in conjunction with a similar optimization/simulation model to obtain real-time operations of a combined power – water system, thus providing a robust solution for cascading failures arising out of contingencies in any of the system.

6. TESTING AN OPTIMIZATION–SIMULATION MODEL FOR OPTIMAL PUMP AND VALVE OPERATIONS WITH REQUIRED STORAGE TANK TURNOVERS

6.1. Summary

An optimization–simulation model is developed for the operation of pumps and valves in water distribution systems (WDS) requiring storage tank turnover. The purpose of the model is to determine the pump and valve operation schedules that minimize the power costs or energy used for pump operations that satisfy demands, pressure, and tank turnover requirements. The modeling approach interfaces a genetic algorithm optimization procedure with the WDS hydraulic and water quality simulator (EPANET) in the framework of an optimal control problem. The interfacing of the genetic algorithm and the EPANET model is implemented within the framework of MATLAB. The application of the optimization–simulation model to a physical water distribution system verifies the importance of such an approach for practical applications.

6.2. Introduction

6.2.1. Water Age and Tank Turnover Rates

Water distribution storage is provided to ensure the reliability of water supply, maintain pressures, equalize pumping and treatment rates, reduce the size of transmission mains, and improve operational flexibility and efficiency (Walski 2000). Storage of finished drinking water in tanks leads to the degradation of water quality through chemical, physical and biological processes that occur as water ages and through external contamination of water in tanks. Water quality problems associated with storage of finished water in tanks include loss of disinfectant residual, formation of disinfection byproducts, development of flavours and odours, increase in pH, corrosion, buildup of iron and manganese, and the occurrence of hydrogen sulfide and leachate from internal coatings. An implicit objective in both the design and operation of distribution system storage facilities is the minimization of detention time and the avoidance of parcels of water that remain in the storage facility for long periods (Grayman and Kirmeyer 2000). The mean detention time within a reservoir is dependent on the inflow and outflow pattern and the volume of water in the reservoir. The age of water in a tank is governed by the volume of water in the tank that is exchanged during daily operations of the WDS.

Tank turnover is the timely replacement of water stored in a tank through consumptive use and pumping. Timely turnover of the water stored in a tank leads to a reduction of water age and improvement in the water quality. Thus, tank turnover rates are an important requirement for efficient operation of a water distribution system (WDS). Water age in storage tanks can be managed through routine turnover through fluctuations in the water levels. Water level fluctuations in a WDS are managed as an integrated operation within pressure zones, demand service areas and the system as a whole rather than on an individual tank basis (American Water Works Association 2002a). This reference discusses available guidelines used for WDS operations for water turnover rates (summarized in Table 6-1).

The research described herein presents a novel method to control WDS pumps and valves for tank turnover considerations which could be used to optimize operation of WDS pumps and valves as well as for the design of new WDS tanks.

Table 6-1 Guidelines for Tank Turnover Rates (American Water Works Association 2002a)

Agency	Guideline
1. Georgia Environmental Protection Division (American Water Works Association 2002a).	1. Daily turnover goal equals 50% of storage facility volume; minimum desired turnover equals 30% of storage facility volume.
2. Virginia Department of Health, Water Supply Engineering Division, Richmond, VA. (American Water Works Association 2002a).	2. Complete turnover recommended every 72 hours.
3. Ohio EPA	3. Required daily turnover of 20%. Recommended daily turnover of 25%.
4. U. S. Navy (1999)	4. Daily turnover of 1/3 of total storage.

6.2.2. Previous Optimization Models for Pump Operation

Goldman et al. (2002) provide a detailed list of optimization models developed for pump operations. Models optimizing pump operations in water distribution systems (WDS) developed after 2002 include: Rao and Salomons (2007); Sakarya and Mays (2000); Salomons et al. (2007); Shamir and Salomons (2008); Ramos and Ramos (2009); Costa et al. (2010); Kurek and Ostfield (2013); Goldman et al. (2002); Hashemi et al. (2014); Ghaddar et al. (2014) and Khatavkar and Mays (2017c).

A few optimization or simulation models for pump operation have been reported in the literature that have been developed as optimization models interfaced with EPANET (Rossman 2000). EPANET (developed by the U.S. Environmental Protection Agency, USEPA) is commonly-used open-source software for hydraulic and water quality simulations. Brion and Mays (1991) present a methodology based on solving large scale nonlinear programming using an optimization-simulation interface. Ormsbee and Reddy (1995) provide a detailed literature review for various optimization-simulation models for pump controls. Sakarya and Mays (2000) developed a model using nonlinear programming interfaced with EPANET for both water quality and hydraulic performance requirements. Goldman and Mays (2005) developed a method that linked simulated annealing with the EPANET model to find optimal pump operation for WDS while meeting both water quality and hydraulic performance requirements. Ozger and Mays (2005) interfaced a simulated annealing model with EPANET for determining the optimal location of isolation valves in water distribution systems for security purposes. Broad et al. (2005) and Salomons et al. (2007) used a surrogate hydraulic model (or meta-model) "trained" to approximate the nonlinear hydraulic equations using a machine learning approach which is a support vector machine that replaces EPANET.

Kang and Lansey (2010) presented a method for real-time scheduling of valve operation and booster disinfection to improve system-wide water quality for known pump operation schedules. The optimization–simulation methodology is based upon linking the EPANET model with a genetic algorithm. Costa et al. (2010) have developed a branch-and-bound algorithm interfaced with EPANET for optimal pump operation of WDS. Kurek and Ostfeld (2013) linked EPANET to a multi-objective methodology using a strength Pareto evolutionary algorithm to demonstrate the tradeoffs between pumping cost, water quality and tank sizing of WDS. Khatavkar and Mays (2018) presented a model for the real-time operation of water distribution systems under limited electrical power availability with consideration for water quality. This method also links the EPANET model with a genetic algorithm.

The main novelty of this paper is consideration of valve controls along with pump controls. The model presented in this paper encompasses tank volume turnover requirements and tank level set points which are important considerations for WDS operations and have never been considered in previous studies.

6.3. Mathematical Formulation of Optimization Model

An optimization model is used to determine the pump and valve operation schedules in order to minimize the total power costs or energy usage for pump operations that satisfy demands, keep nodal pressure within bounds and satisfy tank turnover requirements. The optimization model for pump and valve operations is formulated for a water distribution system (WDS) with K nodes, M pipes, Pu pumps and S tanks. The decision variables for the model include binary control variables for pumps ($X_{p,t}$ = 0 for pump p switched off at time t and 1 for pump switched on) and inflow valve controls ($X_{in(s,t)}$ = 0 for input valve to tank s closed at time t and 1 for valve open) and outflow valve controls ($X_{out(s,t)}$ = 0 for output valve from tank s closed at time t and 1 for valve open) for tanks in the WDS.

6.3.1. Objective function

The total power costs for pumping in a WDS are computed using the following objective function (Obj) that minimizes the power and costs of pumping:

$$\text{Minimize Obj} = \sum_{p=1}^{p=Pu} \sum_{t=1}^{t=T} \left[X_{p,t} \frac{\emptyset_t Q_{p,t} H_{p,t}}{3956 \eta_{p,t}} \right] \qquad \forall p = 1,\ldots,Pu \text{ and } t = 1,\ldots,T \qquad (6\text{-}1)$$

Where, p is the index for pumps, t is the index for time, $\emptyset_t$ is the price of power at time t (\$/HP), $Q_{p,t}$ flow through pump p at time t (gpm), $H_{p,t}$ is the total dynamic head for pump p at time t (ft) and $\eta_{pu,t}$ is the efficiency of pump p at time t. Obj is subject to the following constraints.

6.3.2. Equations Solved Implicitly by EPANET for Hydraulic Analysis

The distribution of flow throughout the network must satisfy the principles of conservation of mass and conservation of energy which are defined as the hydraulic constraints. The conservation of mass at each junction node, assuming water is an incompressible fluid, is

$$\sum_i (Q_{i,k})_t - \sum_j (Q_{k,j})_t - Q_{k,t} = 0 \qquad \forall k = 1,\ldots,K \text{ and } t = 1,\ldots,T \qquad (6\text{-}2)$$

Where, $(Q_{i,k})_t$ is the flow in the pipe m connecting nodes i and k at time t (gpm), and Q_{kt} is the flow consumed (or supplied) at node k at time t (gpm);

The conservation of energy for each pipe m connecting nodes i and j, in the set of all pipes, M, is:

$$H_{i,t} - H_{j,t} = f(Q_{i,j})_t \qquad \forall\ i, j \in K \text{ and } t = 1,\ldots,T \qquad (6\text{-}3)$$

Where, $H_{i,t}$ and $H_{j,t}$ are hydraulic heads at consecutive nodes i and j.

The total number of hydraulic constraints is (K + M) T and the total number of unknowns is also (K + M) T, which are the discharges in M pipes and the hydraulic grade line elevations at K nodes. The pump operation problem is an extended period simulation problem. The height of water stored in a tank s for the current time period t, $y_{s,t}$, is a function of the height of water stored from the previous time period:

$$y_{s,t} = f(y_{s,t-1}) \qquad \forall\ s = 1,\ldots,S \text{ and } t = 1,\ldots,T \qquad (6\text{-}4)$$

The bounds on the level of water storage in a tank s for time t are:

$$\underline{y}_s \le y_{s,t} \le \overline{y}_s \qquad \forall\ s = 1,\ldots,S \text{ and } t = 1,\ldots,T \qquad (6\text{-}5)$$

Where, $\underline{y}_{s,t}$ and $\overline{y}_{s,t}$ are the lower and upper bounds of the elevation of water stored in node s at time t, $\overline{y}_{s,t}$.

Equation 6.5 imposes lower and upper bounds on tank levels based on the tank design. These limits are normally due to physical limitations and fire flow storage requirements of the storage tank. The hydraulic constraints given in equations 6.1 - 6.5 are solved implicit to the optimization model using EPANET (Rossman, 2000) and are documented in detail by Khatavkar and Mays (2018).

6.3.3. Pressure Bound Constraints

One of the major requirements of WDS operations is to maintain pressures within the mandated bounds to ensure public health and the safety of the WDS components. Lower and upper bounds for the nodal pressures in the WDS are imposed using the following constraint as given by Khatavkar and Mays (2018):

$$\underline{P} \leq P_{k,t} \leq \overline{P} \qquad \forall k = 1,\ldots,K \text{ and } t = 1,\ldots,T \qquad (6\text{-}6)$$

Where, $\underline{P}$ is the lower bound for nodal pressures in the system, $P_{k,t}$ is the pressure at node k and time t, and $\overline{P}$ is the upper bound for nodal pressures in the system.

Hydraulic heads ($H_{i,t}$) at nodes are modeled as a function of flow ($Q_{i,j}$) between the nodes in EPANET. Equation 6.3 represents this relationship used by EPANET between the hydraulic heads and flows. Equation 6.6 gives the pressure constraint that imposes lower and upper bounds on the nodal pressures within the WDS, that are not imposed in EPANET. Hydraulic head measures the total energy available at a particular node in the WDS including the datum head, velocity head and pressure head, while the nodal pressures ($P_{k,t}$) represent the pressure head in terms of pounds per square inch (psi) or Pascals (Pa).

6.3.4. Tank Level Set Point Constraints

Water utilities may require the water tanks in a WDS to be filled up to a certain required upper level ($\overline{y_{req\,(s)}}$) and emptied to a certain required lower level ($\underline{y_{req\,(s)}}$) during a 24 h operation of the WDS. These set-points are required for public health and safety and water age considerations. The constraints for tank level set-point requirements are:

$$\max\left([y_{s,t}]_{t=0:T-24}^{t=24:T}\right) \geq \overline{y_{req\,(s)}} \qquad \forall\ s = 1,\ldots,S \text{ and } t = 1,\ldots,T \qquad (6\text{-}7)$$

$$\min\left([y_{s,t}]_{t=0:T-24}^{t=24:T}\right) \leq \underline{y_{req\,(s)}} \qquad \forall\ s = 1,\ldots,S \text{ and } t = 1,\ldots,T \qquad (6\text{-}8)$$

Where $\overline{y_{req\,(s)}}$ and $\underline{y_{req\,(s)}}$ are the required upper and lower set points respectively for tank levels in the system.

Equation 6.7 is a constraint that requires the 24 h maximum water level in a tank to exceed a certain upper set point ($\overline{y_{req\,(s)}}$). Equation 6.8 is a lower tank level set point constraint that requires the 24 h minimum water level in the tank to fall below the lower tank level set point ($\underline{y_{req\,(s)}}$).

6.3.5. Tank Turnover Constraints

Tank turnover can be simply defined as the volume of stored water in a tank replaced through daily consumptive use and inflow. The daily consumptive use of the stored water in a tank can be determined as the sum of outflows from the tank. This study considers that each water supply tank is provided with an inflow and an outflow valve with binary controls $X_{in\,(s,t)}$ and $X_{out\,(s,t)}$. $X_{in\,(s,t)}$ and $X_{out\,(s,t)}$ take the value 1 when the particular valve is to be opened and 0 when the valve is to be closed. The constraints for ensuring the required turnover rate are:

$$\sum_{t}^{t+24} X_{out\,(s,t)} Q_{out\,(s,t)} \geq \theta_s \max\left(\left[Vol_{s,t}\right]_t^T\right) \quad \forall\ s = 1,\ldots,S \text{ and } t = 1,\ldots,T \tag{6-9}$$

$$\sum_{t}^{t+24} X_{out\,(s,t)} Q_{out\,(s,t)} \leq \sum_{t}^{t+24} X_{in\,(s,t)} Q_{in\,(s,t)} \quad \forall\ s = 1,\ldots,S \text{ and } t = 1,\ldots,T \tag{6-10}$$

Where, $Q_{in\,(s,t)}$ and $Q_{out\,(s,t)}$ are inflow and outflow for tank s at time t, $Vol_{s,t}$ is the volume of water stored in tank s at time t, and

θ_s is the mandated turnover rate for the tank s.

6.4. Reduced Optimization Model

The optimization model presented in Equations 6.1–6.10 is an optimal control problem solved by interfacing an optimization model (genetic algorithm) with an EPANET simulator. The genetic algorithm scores the various solutions in a solution set with respect to the extent to which the model achieves constraint compliance. The scoring is done through a fitness function (equation 6.11) based on a reduced form of the full optimization problem given in equations 6.1–6.10. A reduced optimization model with constraints in the form of penalty functions is solved by the genetic algorithm, which solves unconstrained problems. The reduced optimization model is:

$$\text{Minimize } Obj_{reduced} = W_1 \sum_{p=1}^{p=Pu} \sum_{t=1}^{t=T} \left[X_{p,t}\, \emptyset_t \frac{Q_{p,t}\, H_{p,t}}{3956\eta_{p,t}}\right] +$$

$$W_2 \sum_{k=1}^{k=K} \sum_{t=1}^{t=T} \left[w_1 \max(0, 0 - P_{k,t}) + w_2 \max(0, \underline{P} - P_{k,t}) + w_3 \max(0, P_{k,t} - \overline{P})\right] +$$

$$W_3 \sum_{s=1}^{s=S} \left[\sum_{1}^{T/24} \max\left(0, \overline{y_{req\,(s)}} - \max([y_{s,t}]_{t=0:T-24}^{t=24:T})\right) +\right.$$

$$\left.\max\left(0, \min([y_{s,t}]_{t=0:T-24}^{t=24:T}) - \underline{y_{req\,(s)}}\right)\right] +$$

$$W_4 \sum_{t=1}^{t=T} \sum_{s=1}^{s=S} \max\left(0, \left[\theta \max\left(\left[Vol_{s,t}\right]_t^T\right) - \sum_{t}^{t+24} X_{out\,(s,t)} Q_{out\,(s,t)}\right]\right) \tag{6-11}$$

Where, $Obj_{reduced}$ is the reduced objective function, W_1 to W_4 are penalty weights associated with objective function, pressure bounds, tank level bounds and tank volume turnover constraints respectively, and w_1 to w_3 are additional penalty weights for negative pressures, lower pressure bound violations and upper pressure bound violations respectively.

The penalty weights (W_1-W_4) are determined through a sensitivity analysis for each application of the optimization model. The sensitivity analyses are performed for all terms involved in the reduced optimization model (Equation 6.11). The value of weights to be used for a particular term depend upon the relative importance of the penalty term and the magnitude of the numeric value of the penalty term. Parameters considered for the sensitivity analyses are the value of the objective function, number of pressure bound violations, number of tank level set point violations and the number of tank volume turnover constraint violations. The optimization model is tested for a range of values for each of the penalty weights and a combination of penalty weights yielding the minimum objective function value and least amount of bound violations for pressures, tank level set point constraints and tank volume turnover constraints is chosen for the application. w_1, w_2, and w_3 impose additional weights on lower negative pressures, lower pressure bounds, and upper pressure bounds respectively. EPANET is a demand-driven simulation model that always meets the demands irrespective of pressures at nodes in the WDS. The solutions may have negative pressures at certain nodes for operations where enough water is not pumped to meet the water demands. For every iteration, the GA provides a new set of decision variables (including pump and valve controls) and an EPANET simulation is performed based on the decision variables. A negative pressure penalty function with a large additional penalty weight (w_1) is used in the reduced optimization model to exclude EPANET solutions with negative pressures. The final solution from the optimization-simulation can never include an operation schedule that results in negative pressures.

6.5. Model Solution Methodology

An interface between the genetic algorithm (WDS optimization model) and the EPANET simulator is used to solve the reduced optimization model given in equation 6.11. The MATLAB–EPANET interface is created from the opensource EPANET-MATLAB toolkit (Eliades 2016). The interface facilitates use of the functionality of EPANET within the MATLAB environment by passing the various commands between the MATLAB mathematical language and the EPANET simulator. This toolkit was used in conjunction with the genetic algorithm in MATLAB to effect the overall optimization–simulation methodology shown in

Figure 6-1.

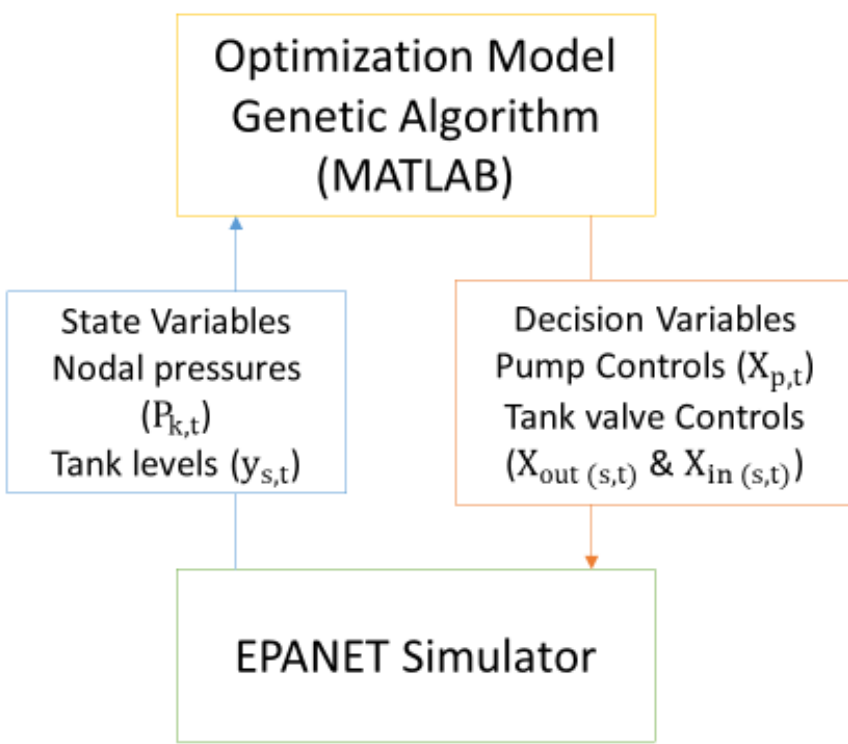


Figure 6-1 Optimization–Simulation Methodology

The genetic algorithm (in MATLAB) searches over the control variable (pump operations, $X_{p,t}$, and valve operations, $X_{in(s,t)}$ and $X_{out(s,t)}$) using the values of the state variables (nodal pressures, pipe flows, water age and tank levels) determined in through an extended time simulation in EPANET. In other words, the optimizer passes pump and valve operations to the simulator which determines the state variables (nodal heads, pipe flows, and tank levels). This process continues until an optimal or near optimal solution is reached.

Basic parameters of genetic algorithm include population size, number of allowable generations, crossover probability and mutation probability. Population size defines the total number of solutions (number of times EPANET is solved) in a particular generation. A generation is a set of solutions, from which the next generation is chosen based on crossovers and mutation. A sensitivity analysis was performed to set the genetic algorithm parameters in this study. The combination of parameters giving the best convergence of solution in minimum computational time was chosen. A population size of 30 was used for the application, with the maximum allowable generations bound at 1000. The crossover probability and the mutation probability were set at 60% and 20% respectively. The solutions converged perfectly after 100 generations (3000 function calls) with a computation time of 3650 seconds.

6.6. Example Application

The example application is the water distribution system (WDS) of city XYZ which has 1427 junctions (nodes), 3 tanks (T-1, T-2 and T-3), 2 treatment plants which are modeled as reservoirs (R-1 and R-2), 1789 pipes, 4 pumps (PMP-1 to PMP-4), 2 pressure release valves (PRV) and 6 flow valves.

Figure 6-2 is a map of the WDS with the locations and details of tanks and pumps in the system. Each tank in the system is given an inflow and an outflow valve. Operation of inflow and outflow valves for the tanks is important for maintaining the required levels in the tanks and for draining the tanks to a certain level on a timely basis to ensure public health and safety. A control valve is provided on the downstream of both the inflow and outflow valves for restraining the direction of flow into and out of the tank. The inlet and outflow valves for the tanks can be controlled in four different combinations, as listed in Table 6-2.

Table 6-2 Control of Inflow and Outflow Valves for Tanks

Inflow Valve Status	Outflow Valve Status	Flow Condition
1. Open	1. Closed	1. Inflow into the tank
2. Closed	2. Open	2. Outflow from the tank
3. Open	3. Open	3. Inflow or outflow depending on the head difference between the tank and the node downstream of the valves. (depending on the pump operations)
4. Closed	4. Closed	4. No flow in or out of the tank

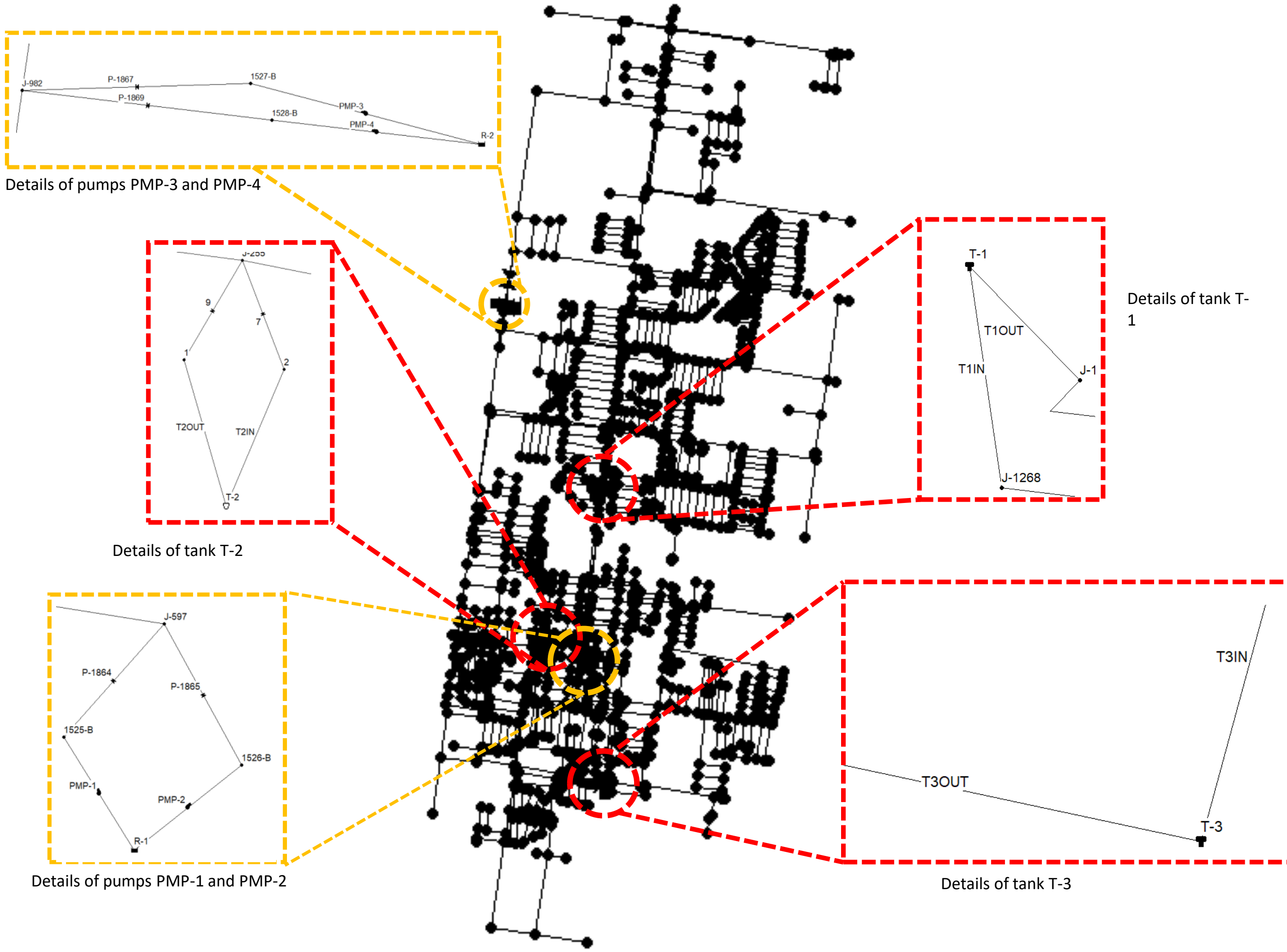


Figure 6-2 Details of the Water Distribution System of City XYZ

Pumps PMP-1 and PMP-2 operate in parallel to pump water from reservoir R1 and pumps PMP-3 and PMP-4 pump water from reservoir R2. Figure 6-3 shows the pump curves used for the four pumps. The pumps are assumed to operate at 100% efficiency.

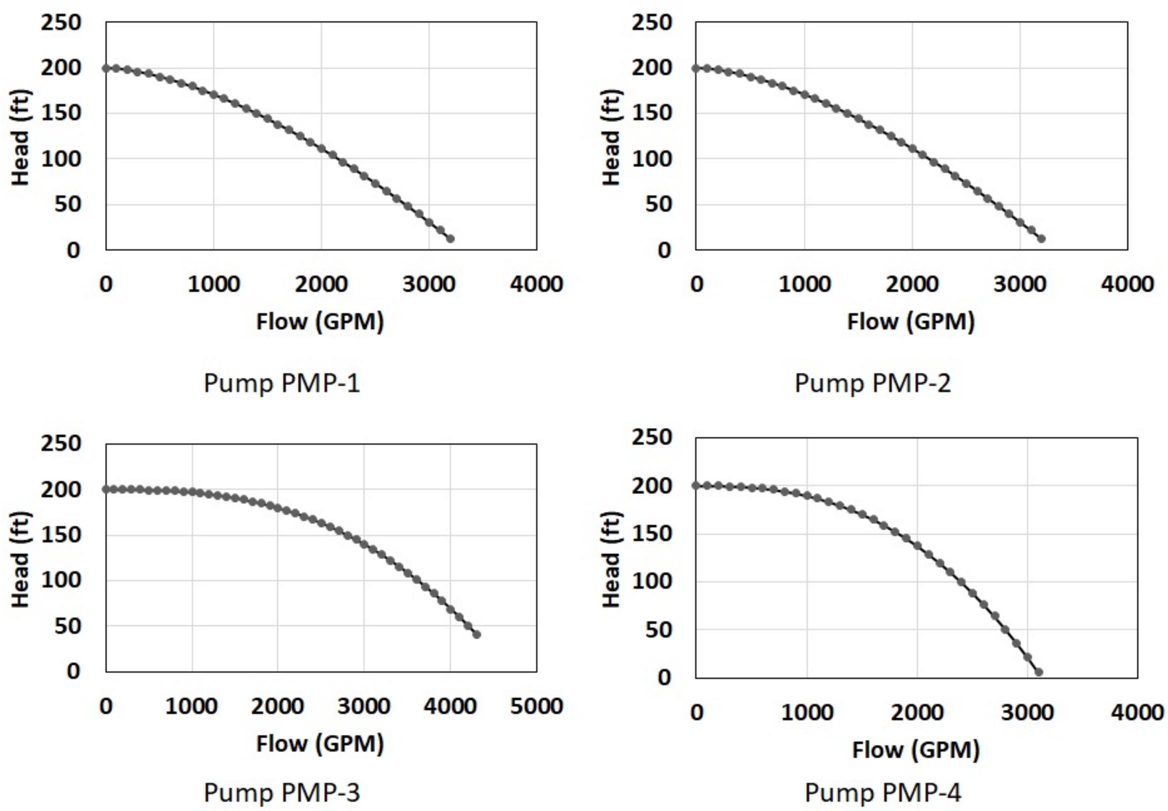


Figure 6-3 Pump Curves

Tanks T-1 and T-2 are modeled as cylindrical tanks with diameters of 38.5 ft (11.74 m) and 40 ft (12.2 m). Both T-1 and T-2 are elevated storage reservoirs with bottom elevations of 205.58 ft (62.67 m) and 214 ft (65.23 m). The overflow elevations of tanks T-1 and T-2 are 234 ft (71.32 m) and 244 ft (74.37 m). Tank T-3 is conical at the bottom and the top is nearly cylindrical; it is modeled using a water level–volume curve. Tank T-3 has a bottom elevation of 214.02 ft (65.23 m) and the overflow elevation of the tank is 251 ft (76.50 m). The overall demand in the WDS follows a diurnal pattern as shown in Figure 6-4.

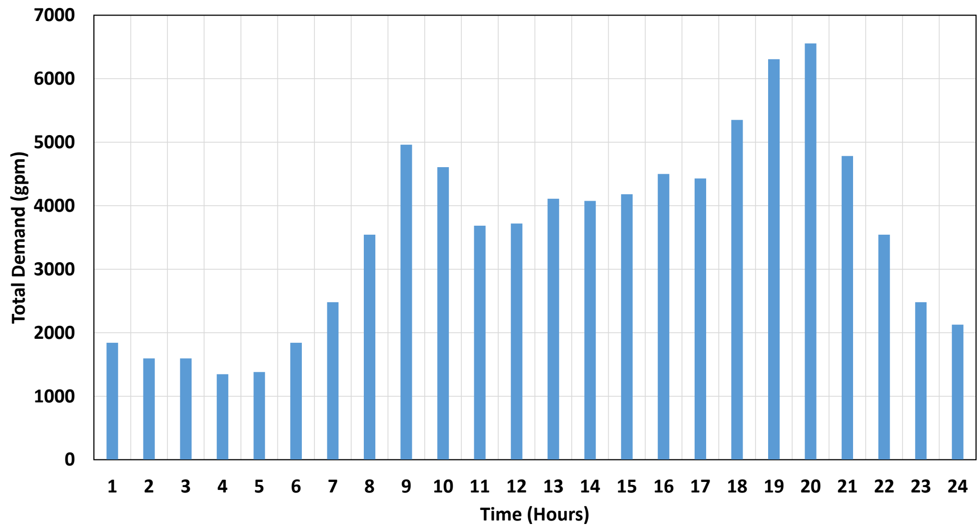


Figure 6-4 Trends of Total Demand for City XYZ

6.7. Application Results and Observations

6.7.1. Model Application

The reduced optimization model, solved by the genetic algorithm (equation 6.11), was used for the pump and valve operations of the example system using the optimization–simulation interface outlined in

Figure 6-1. A simulation time of 14 d (336 h) was used in EPANET. Two scenarios (both a 25% and a 40% turnover rate for WDS operation) were considered for this application, based on the tank turnover guidelines given in Table 6-1. The lower and upper pressure boundaries for the WDS used to optimize the pump and valve operations were 30 psi (206.85 kPa) and 80 psi (551.58 kPa).

Even though the results use the overflow elevations of the tanks for pump operation, a maximum high-water elevation should be established that is less than the overflow elevation for each tank. In a normal operation the utility would never want to fill tanks to the overflow elevation, but to some specified maximum high-water level a few feet or meters below the overflow elevation. This application did not require emergency storage volumes for each time period but the minimum levels in tank T-3 provide a large emergency storage quantity available throughout each 24 h cycle.

Figure 6-5 shows the changes in water levels (heads) in the three tanks for a 25% tank turnover. The tanks fill up during the early hours of the day (off-peak demand hours) and drawdown during the later hours of the day. All three tanks fill to the overflow elevations. The 24 h trends for the simulation were observed to cycle (repeat) for a 24 h simulation period after the third day (72 h) of the application. Keep in mind that a maximum tank operating level would be established for each tank, not allowing it to fill above that level. Figure 6-6 shows the change of water levels in the three tanks for a 40% tank turnover. Tanks T-1 and T-2 both fill to their respective overflow elevations and tank T-3 fills to 248 ft (75.60 m), 3 ft (0.91 m) below the overflow elevation. The elevation of 248 ft (75.60 m) could be the maximum operating level for tank T-3, to provide a safety margin in the case of altitude failure or other failures that might occur in the system. Figure 6-7 and Figure 6-8 show the trends of water age in the three WDS tanks for the 2 week (336 h) extended period simulation for the 25% and 40% turnover rates. For the 25% turnover scenario (Figure 6-7), the highest water age observed during the 2-week simulation is ~90 hours and the water age is seen to be stable after the first week of simulation..

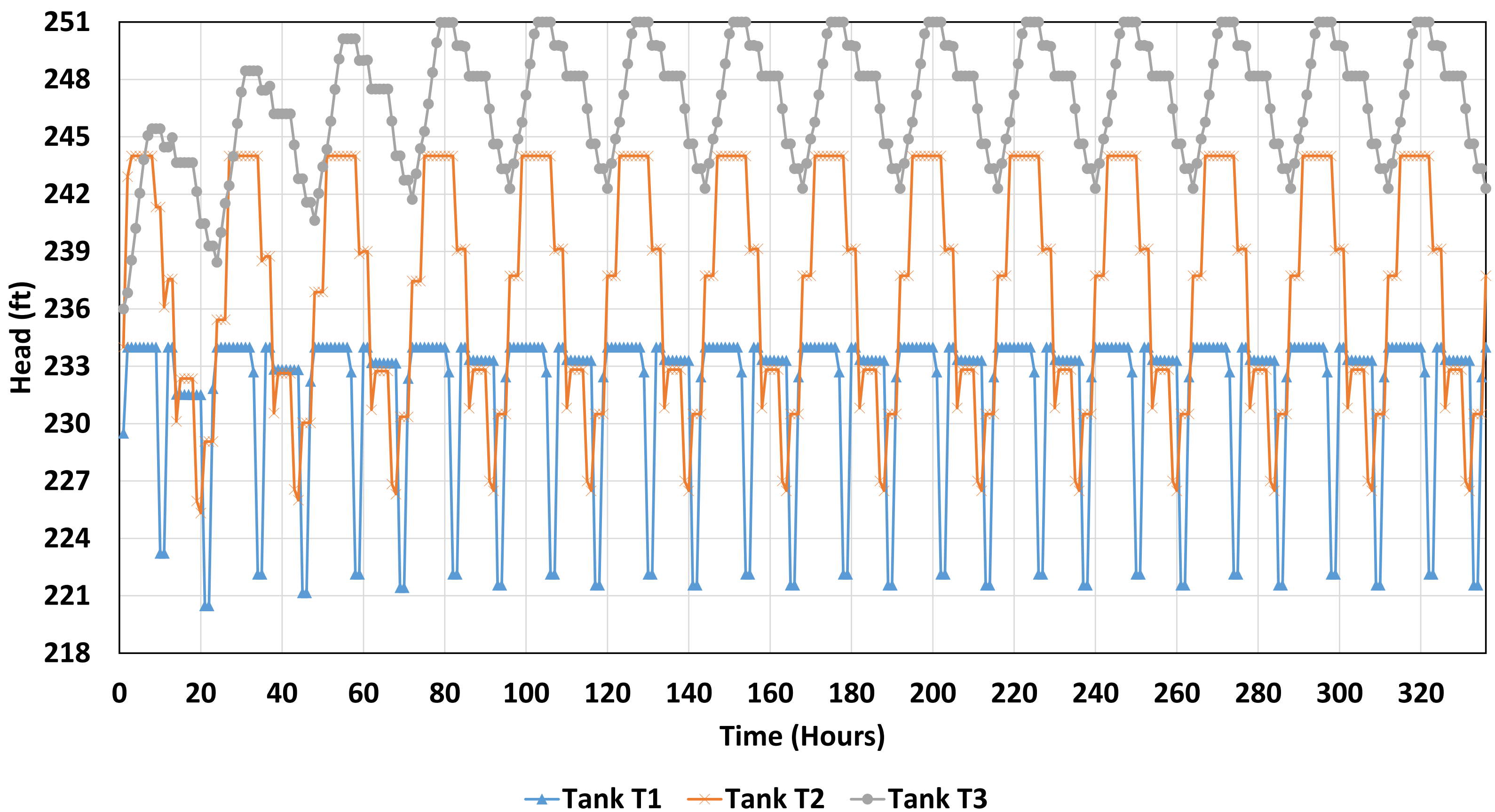


Figure 6-5 Storage Tank Levels for 25% Turnover

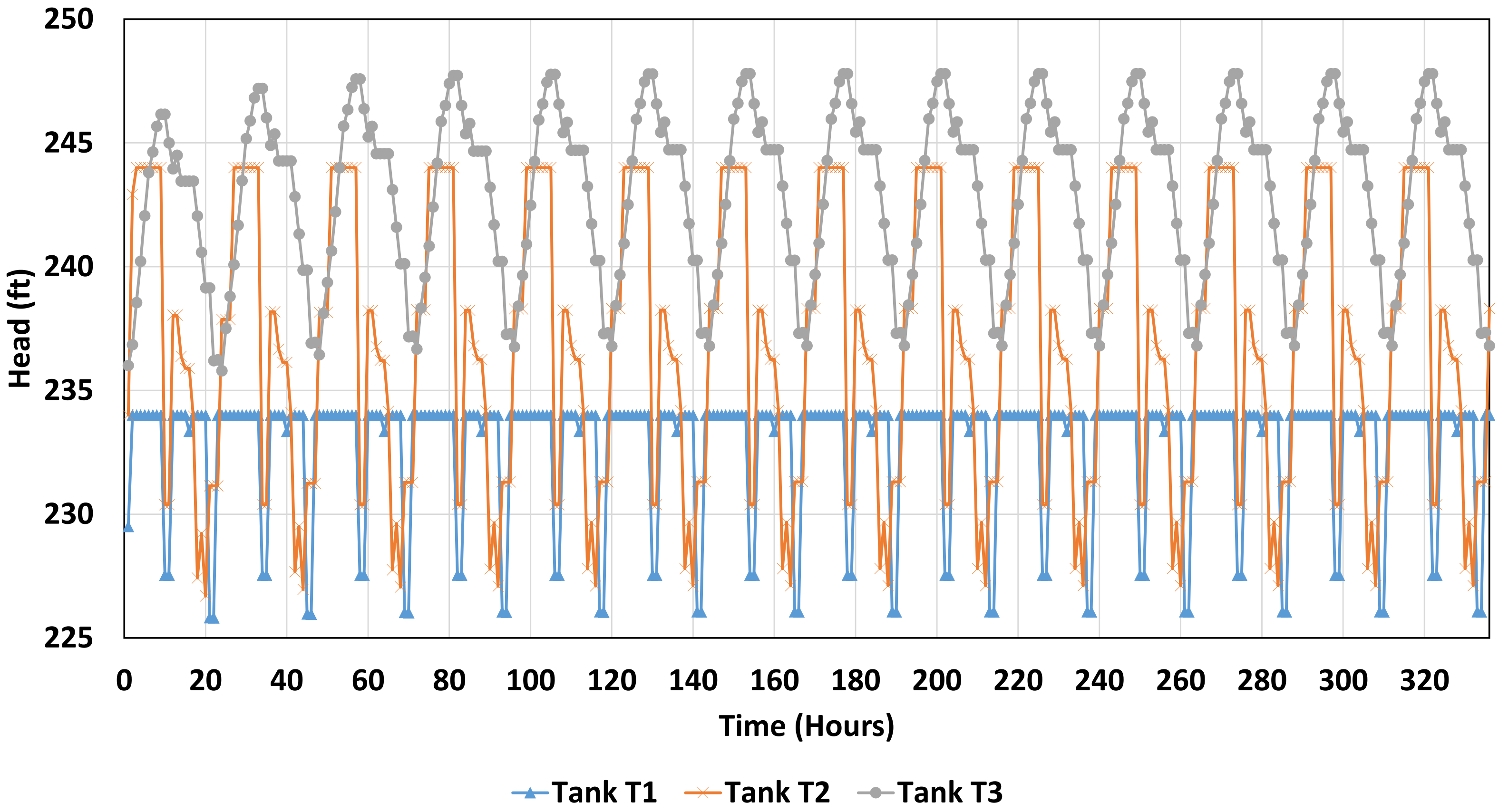


Figure 6-6 Storage Tank Levels for 40% Turnover

For the 40% turnover, the highest water age observed during the 2-week simulation is ~70 hours. The difference between the maximum water ages for the two scenarios show that the tank turnover requirements can affect the water age in the system. In both the scenarios, the water age in the tank stabilizes within the acceptable limit of 90 h prescribed by the American Water Works Association (American Water Works Association 2002b). The lower and upper pressure bounds used to optimize the pump and valve operations, 30 psi (206.84 kPa) and 80 psi (551.58 kPa), were satisfied for both the 25% and 40% turnovers.

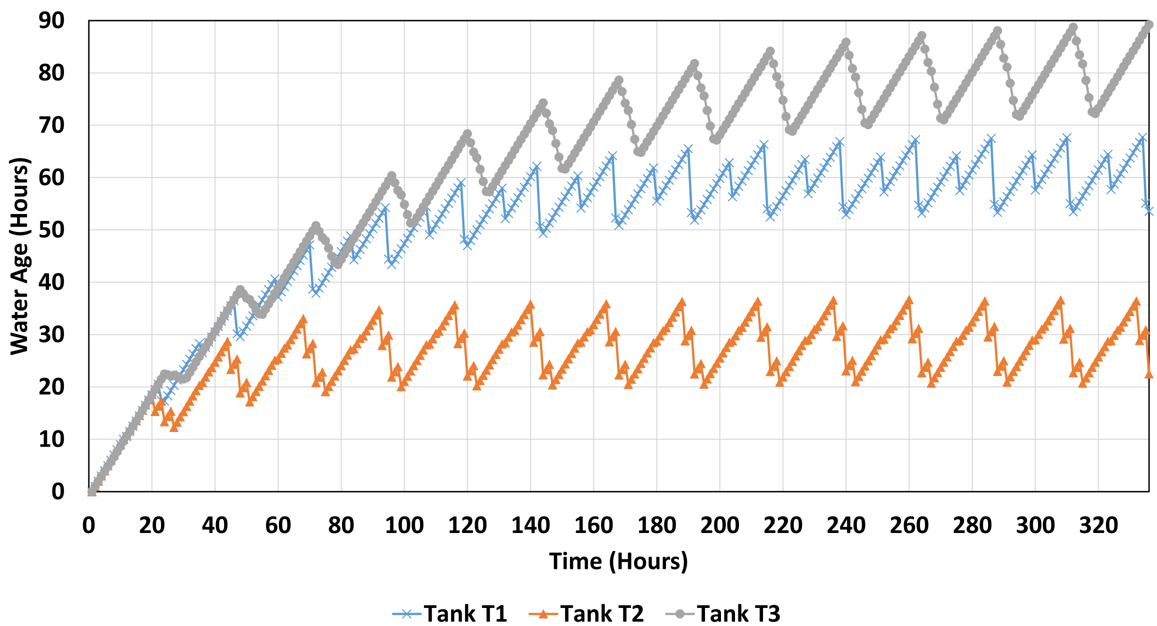


Figure 6-7 Water Age in Tanks for 25% Turnover Each Day

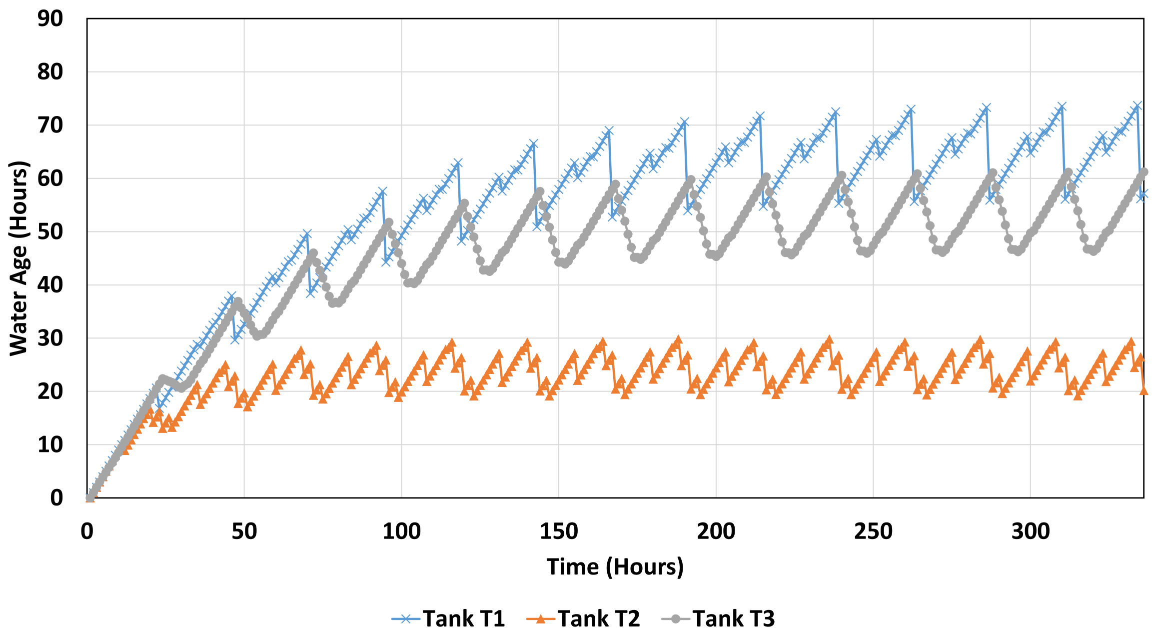


Figure 6-8 Water Age in Tanks for 40% Turnover Each Day

6.7.2. Modifications to WDS of City XYZ

To further test the new optimization–simulation methodology, the WDS of city XYZ is modified by adding an extra pump (with pump curve similar to pump PMP 4, as shown in Figure 6-3) and increasing the size of tank T-1 from 38.3 ft (11.67 m) diameter to 68.3 ft (20.82 m) diameter. These modifications are alternatives being considered for modification of WDS for City XYZ along with the modifications detailed in Sections 5.3 and 5.4. The elevations of tank T-1 remain the same with a bottom elevation of 205.58 ft (62.66 m) and an overflow elevation of 234 ft (71.32 m). The resulting tank water levels for a 25% turnover are shown in Figure 6-9. Water levels in tanks for the optimized operation show that all three tanks fill to their overflow elevations, even though in practice operators would not fill the tanks to that level. It takes ~7d–8 d for the system to stabilize such that the levels repeat themselves each day. The last 24 h of operation would be used as the actual operation schedule for the pumps and valves for the 25% turnover. Water age in the 3 tanks of the modified WDS of city XYZ for a minimum 25% turnover are shown in Figure 6-10. Keep in mind that if we did not allow the tanks to fill to the overflow level, but to a lower elevation (e.g. the maximum fill level at 248 ft (75.60 m) in tank T-3, then the water ages would be even lower for that tank.

6.7.3. Tank T-1 Overflow Elevation Changed to Tank T-3 Elevation

Now both the overflow elevations of tanks T-1 and T-3 are increased to 251 ft (76.50 m). A boundary of 25%–35% turnover rates is specified in the optimization. The resulting tank levels are shown in Figure 6-11 and the water age is shown in Figure 6-12. Even though we have considered the overflow elevation for the operation of each tank, a lower maximum operating elevation would be used in practice. For example, a maximum operating level for tanks T-1 and T-3 could be set at 248 ft (75.60 m) and for T-2 could be set at 241 ft (73.45 m). This would allow for plenty of emergency storage in the system and would allow for a large turnover.

All the three tanks fill up to their overflow elevations and empty out to the required turnover volume during every 24 hours of operation. Tank T-1 was observed to fill up to the maximum elevation of 251 ft (76.50 m). Maximum turnover was observed in tank T-2, followed by tank T-1 and T-2. All the storage tanks in the system were observed to show a turnover of atleast 25% for every 24 hours of operation. The water age in the three storage tanks (shown in Figure 6-12) is under the acceptable limit of 90 h. Larger volume turnovers were observed in tank T-1 as compared to storage levels shown in Figure 6-9. The water age in tank T-1 (shown in Figure 6-12) is noted to have increased as compared with Figure 6-10 due to the larger storage volume of the tank.

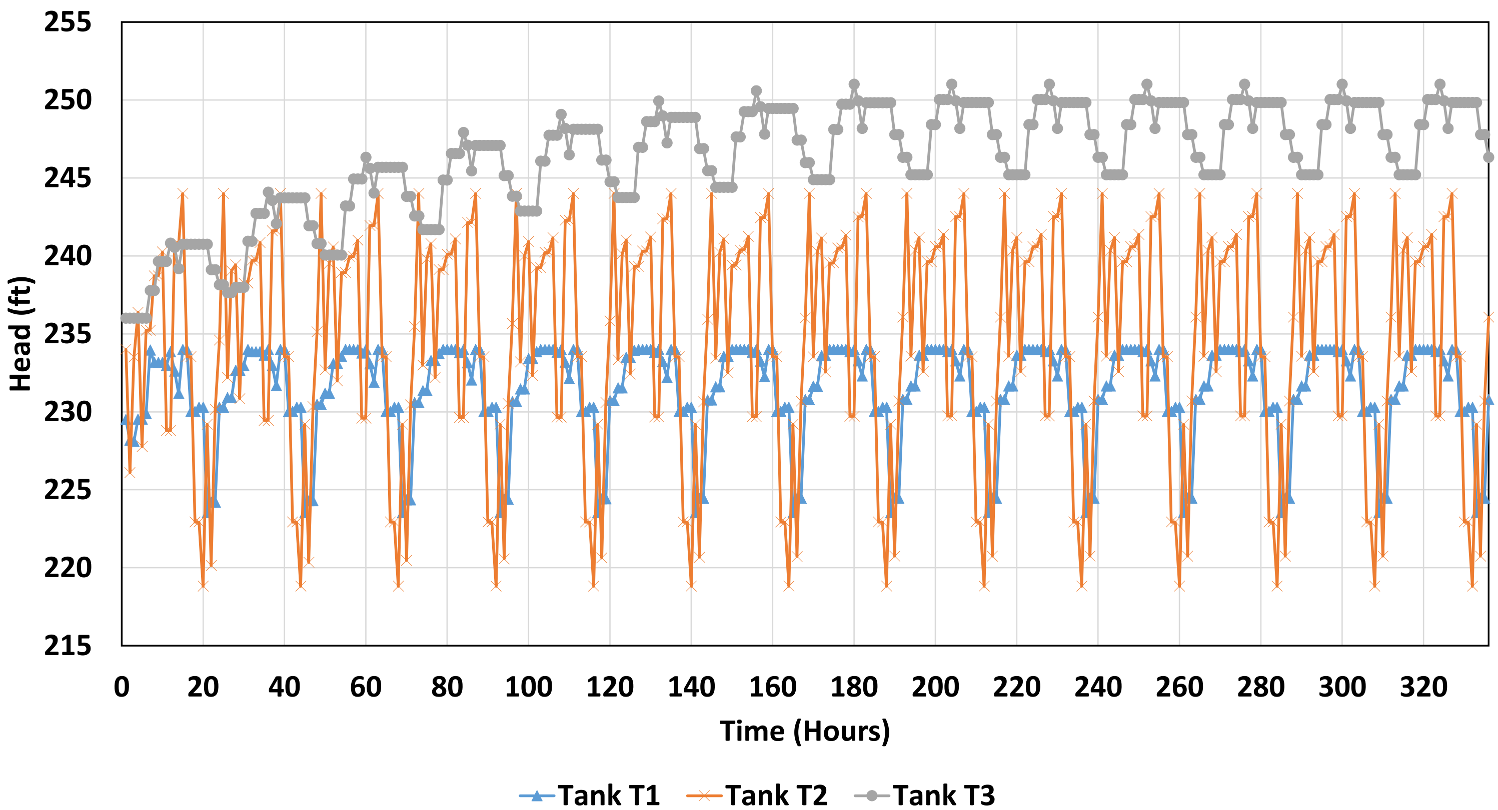


Figure 6-9 Storage Tank Levels for 25% Turnover for the Modified WDS of City XYZ

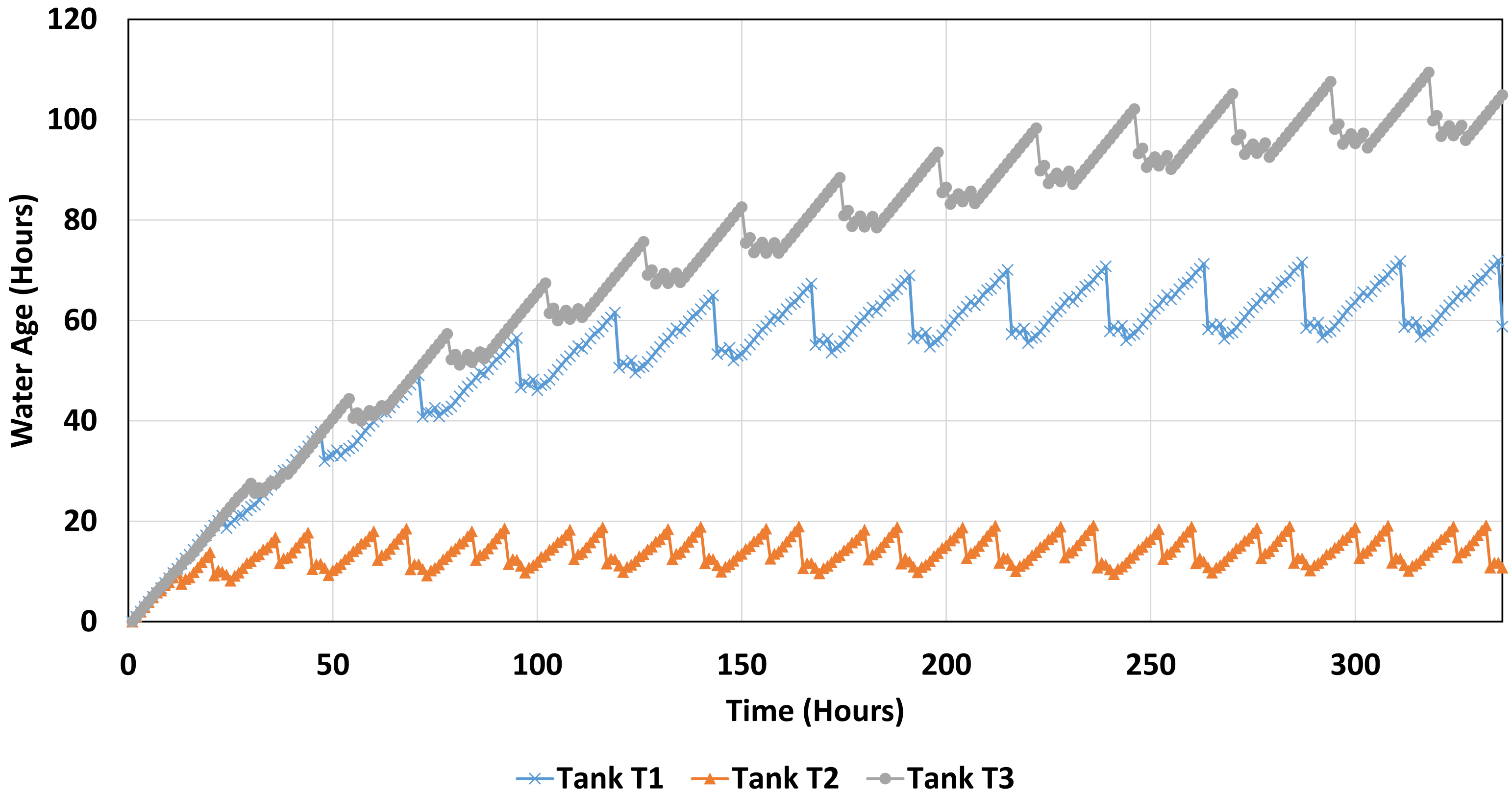


Figure 6-10 Water Age in Tanks of the Modified WDS of City XYZ for 25% Turnover Each Day

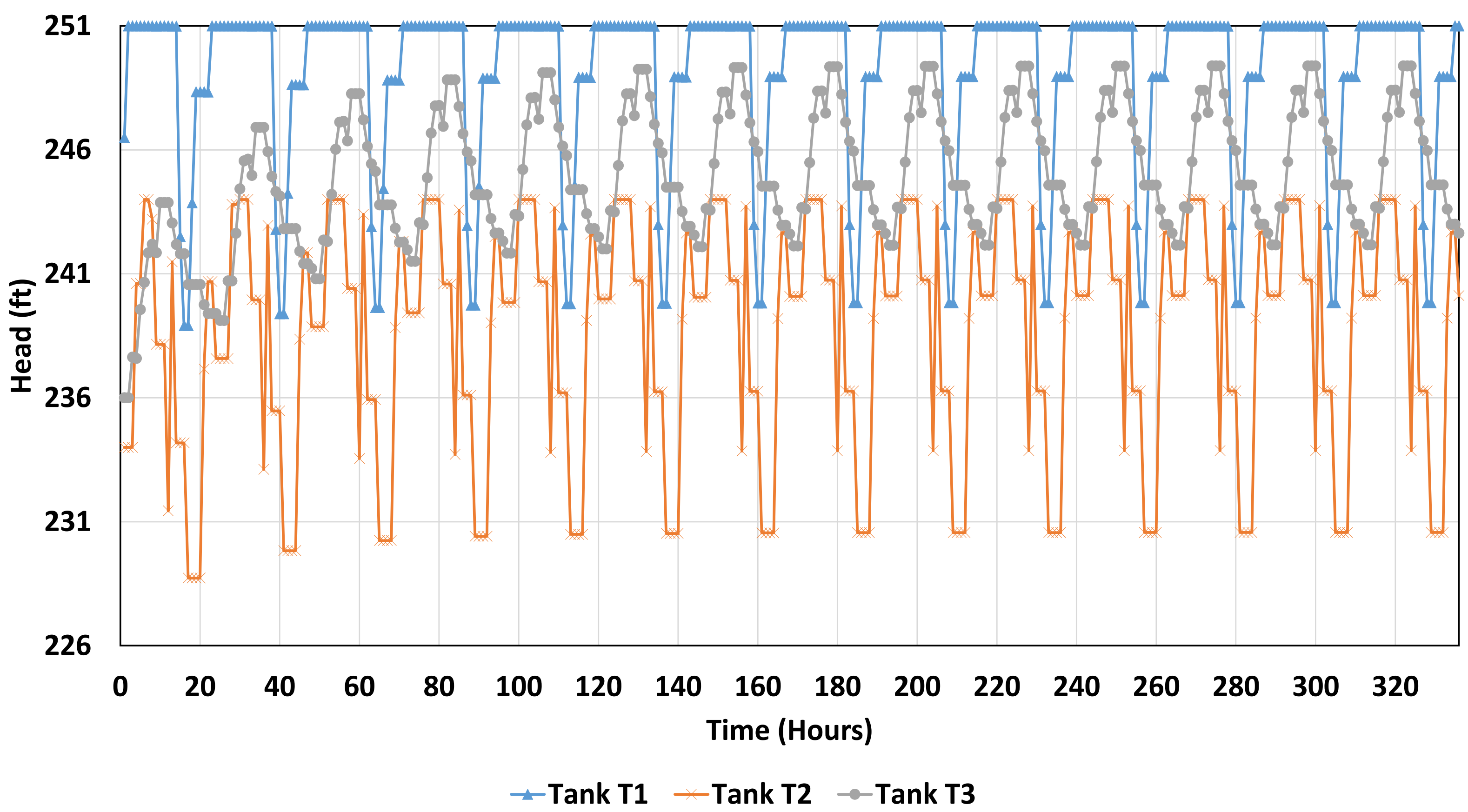


Figure 6-11 Storage Tank Levels for a Minimum 25% Turnover for the Modified WDS of City XYZ with Tank T-1 Raised to the Elevation of Tank T-3

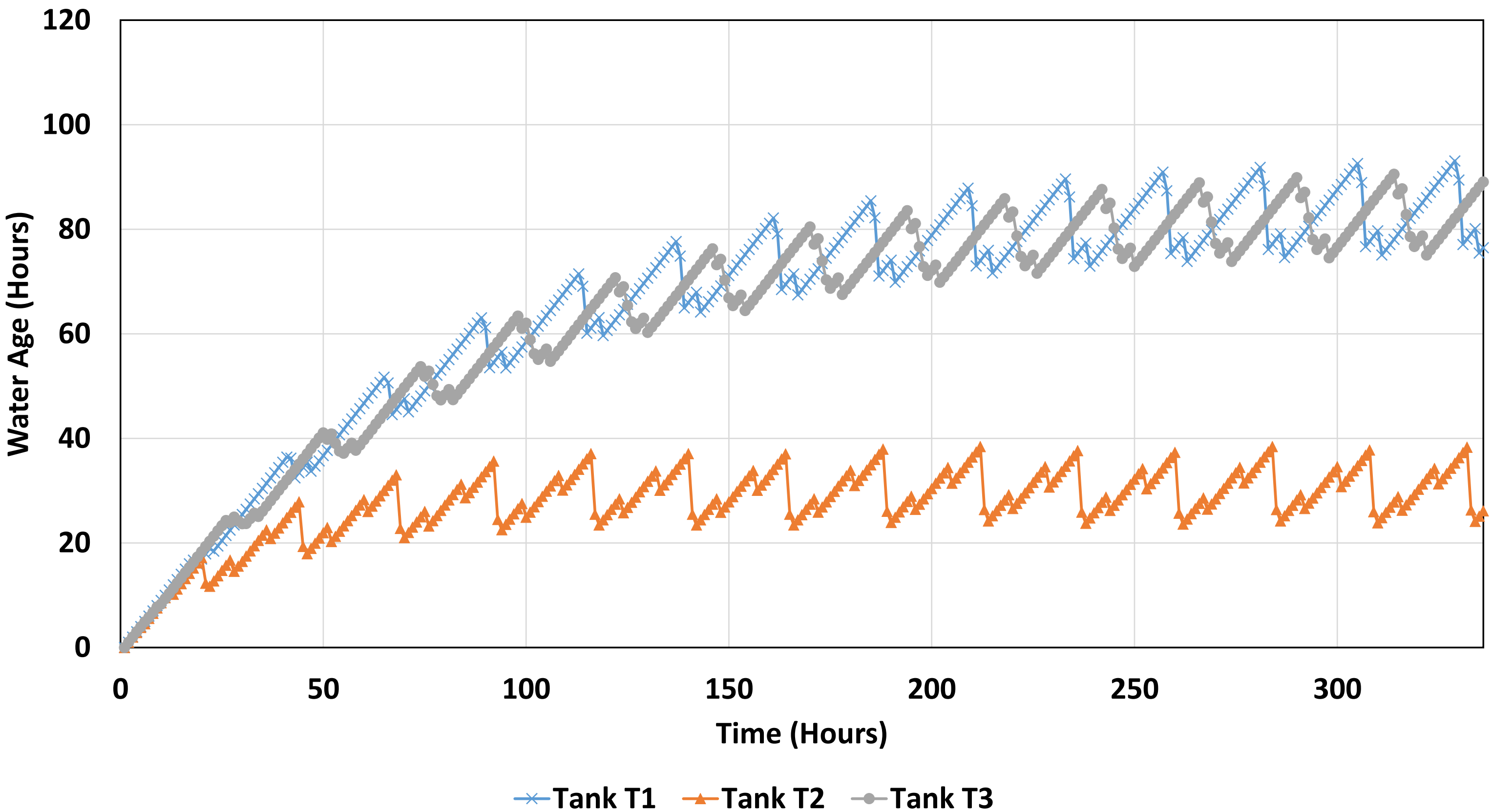


Figure 6-12 Water Age in Tanks of the Modified City XYZ WDS for a Minimum 25% Turnover with Tank T-1 Raised to the Elevation of Tank T-3

Raising the T-1 overflow elevation to T-3 elevation provides adequate emergency storage for all the three tanks throughout the simulation

6.7.4. Maximum Operating Water Levels Set at 248 ft (75.60 m) in Tanks T-1 and T-3

To make the resulting operation more realistic, the maximum operating water levels were set at 248 ft (75.60 m) in tanks T-1 and T-3. The resulting tank levels for the operations are shown in Figure 6-13. Note that T-1 and T-3 both fill to the maximum operating levels of 248 ft (75.60 m). The water ages in each of the tanks as a function of time are shown in Figure 6-14. The results for this application are similar to those observed in Figure 6-11 and Figure 6-12, except for the lower maximum water levels in tank T-1 due to the lower maximum operating water level setting. The water age and emergency storages are within the acceptable limits throughout the 336-h simulation.

6.8. Conclusions

The model presented in this chapter is a novel approach to optimization of daily pump operation schedules and valve controls considering the tank turnover requirements for a WDS. In addition to minimizing the power costs associated with pumping required in a WDS, the model also achieves compliance of the system operations with tank turnover requirements by optimizing the pump and valve operations.

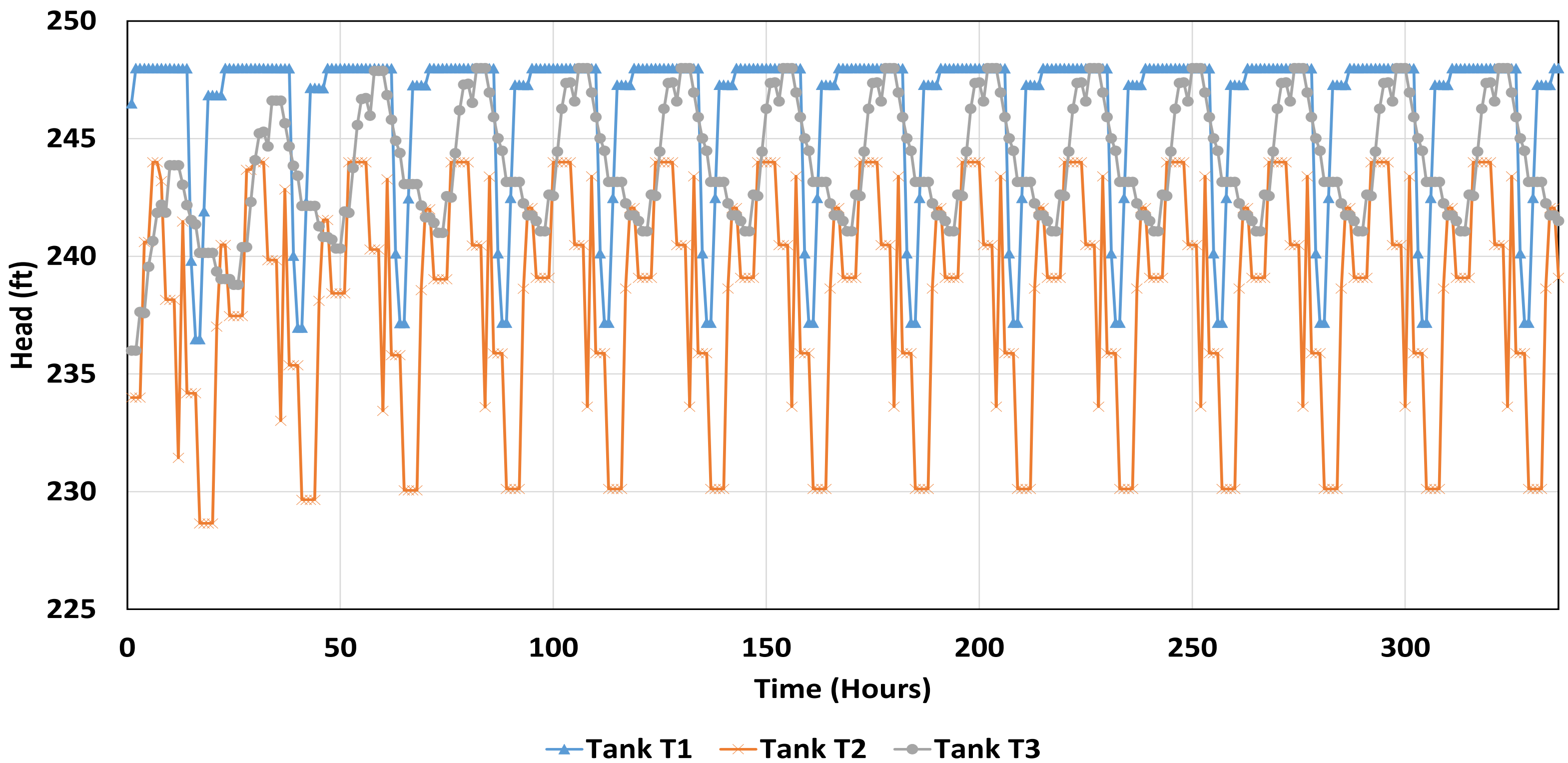


Figure 6-13 Storage Tank Levels for Maximum Operating Levels in Tanks T-1 and T-3 set at 248 ft (75.60 m)

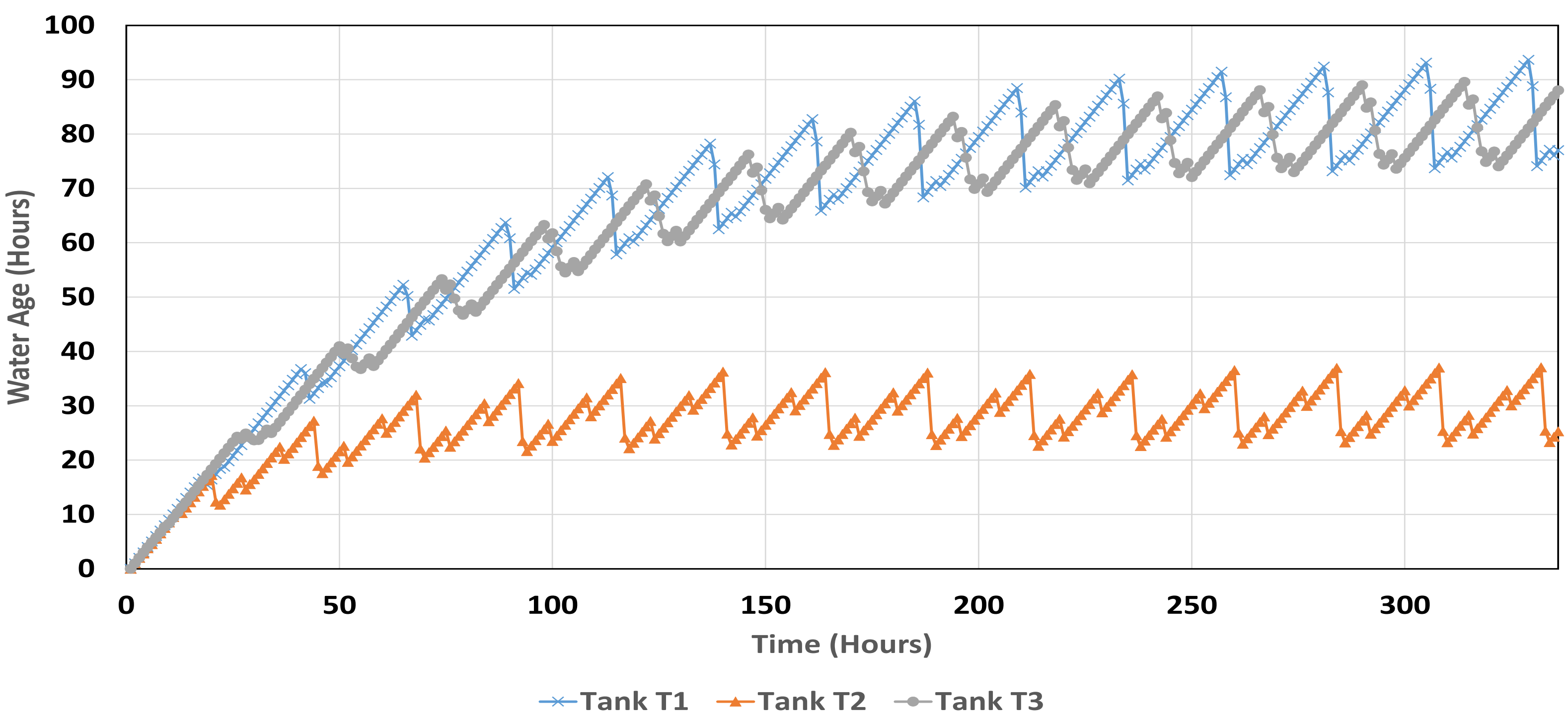


Figure 6-14 Water Age in Tanks for Maximum Operating Levels in Tanks T-1 and T-3 Set at 248 ft (75.60 m)

7. RESILIENCE OF WATER DISTRIBUTION SYSTEMS DURING REAL-TIME OPERATIONS UNDER LIMITED WATER AND/OR ENERGY AVAILABILITY CONDITIONS

7.1. Summary

A new methodology for determining system operation resilience is presented for the real-time operation of water distribution systems (WDS) under critical conditions of limited water and/or limited electrical energy resulting from extreme drought or electric grid failure. Resilience for water distribution systems is defined as how quickly the WDS recovers or bounces back from emergency operations to normal operations. The algorithm for operational resilience is interfaced with an optimization-simulation model for the real-time optimal operation of water distribution systems. The resilience methodology considers both demand and water quality requirements of both the municipal WDS and the power plant cooling systems. The optimization-simulation modeling approach interfaces a genetic algorithm optimization procedure with the WDS hydraulic and water quality simulator (EPANET) in the framework of an optimal control problem. The interfacing of the genetic algorithm in MATLAB and the EPANET model is implemented using a MATLAB – EPANET toolkit. An example WDS including two cities, five power plants and reclaimed water from a waste-water treatment plant is used to demonstrate the application of the system operation resilience concepts to assess the performance. Applications of the methodology are used to illustrate improved operation resilience of the system.

7.2. Introduction

Power distribution systems (PDS) and the water distribution systems (WDS) are two critical infrastructure systems with operational interdependencies that have been recognized and studied in broad generalities for some time. The main dependency of the PDS on the WDS is the requirement of water for the cooling cycle of thermoelectric power generation. The main dependency of the WDS on the PDS is the power for pumping water from the source to the treatment plant through the WDS. The interdependencies become more acute under conditions of limited water and/or power availability. The optimization-simulation framework used for this study considers these interdependencies. If power plant cooling water demands are not met, the PDS may fail to supply adequate power for WDS pumping stations leading to a cascading failure of both the systems. This study presents a new methodology for computation of operational resilience of WDS considering the interdependencies between the water and power distribution systems. The methodology can consider both the impacts of reduced water availability and reduced power supply simultaneously or individually.

7.2.1. <u>Definition of operational resilience for water distribution systems</u>

Several definitions of resilience of an engineered system are present in the literature, a few notable ones are presented in this section. Resilience measures the ability of a system to "bounce back" following some disturbance or failure. Following are general definitions of resilience not necessarily related to water distribution systems. Folke (2006) added that resilience measures the capacity of an engineered system to absorb disturbance while the system is undergoing changes to retain essentially the same function, structure, and feedbacks. Bruneau et al. (2003) defined resilience through the four R's of resilience including robustness, redundancy, resourcefulness and rapidity. Walker el al. (2016) acknowledge that different interpretations of resilience used in the literature cause confusion and state that resilience of a system needs to be considered in terms of the attributes that govern a system's dynamics. Pandit and Crittenden (2012) state that resilience is the capacity of a system to absorb shocks and perform under perturbations and that resilience can serve as an appropriate indicator of system performance. Hosseini et al. (2016) provide a detailed review of definitions and measures of resilience for engineered systems. Shin et al. (2018) investigated into the definitions of quantitative resilience measures for water distribution systems.

A few definitions of resilience for WDS network structure and operations have been reported in the literature. Blackmore and Plant (2008) focused on the risk-based definition of resilience for WDS operations to provide a methodology for resilient design of WDS. Hashimoto et al. (1982) defined resilience for water distribution systems as how quickly the WDS recovers or bounces back from failure once failure has occurred. Baños et al. (2011) present a resilience index for WDS performance for uncertainty-based design of WDS. Pandit and Crittenden (2012) explain an index of network resilience for urban WDS for determining and evaluating alternative configurations for the WDS. Zhuang et al. (2013) presented a methodology for resilience of WDS considering failure recovery times. Other similar studies for resilience-based design of WDS include Herrera et al. (2016); Sweetapple et al. (2018) and Ma et al. (2018).

Most of the studies in the literature focus on the resilience-based design of WDS except a couple of studies by Aydin et al. (2014 a and b), which are based on operational resilience of WDS. Operational resilience is defined as the capacity of an engineered system to absorb disturbance and reorganize while undergoing change so as to still retain essentially the same function, structure, identity and feedbacks (Walker et al., 2004; Blackmore and Plants, 2008). Operational resilience of water distribution systems is defined by Aydin et al. (2014 a) as how quickly a WDS recovers from not meeting acceptable pressure levels. Aydin et al. (2014 b) defined operational resilience as the ratio of the number of times a satisfactory performance follows an unsatisfactory performance and the number of times an unsatisfactory performance occurs within the system. This study uses the definition of operational resilience given by Aydin et al. (2014 b) for developing a real-time resilience assessment methodology for WDS operations under emergency conditions. An unsatisfactory performance would result when one of the performance conditions such as pressure requirements are not met. Meeting the end user water quality requirement is another important performance criterion for WDS operations.

7.2.2. Infrastructural – Operational Resilience (IOR)

New metrics and a new methodology for determining system infrastructural-operational resilience (IOR) are presented for the optimal real-time operation of water distribution systems (WDS) under critical conditions of limited water and/or limited electrical energy resulting from extreme drought or electric grid failure. IOR is calculated by weighting the operational resilience with the appropriate infrastructure robustness metrics. The IOR computation method utilizes results from a combined optimization-simulation framework which involves capturing the interactions and associated dynamics between the power distribution system (PDS) and WDS. A realistic example of the WDS and PDS is used to demonstrate the application of the resilience concepts to assess the WDS performance under interdependent operations of the two systems.

7.2.3. Optimization – Simulation Approach

Khatavkar and Mays (2018) provide a detailed description of the optimization - simulation model for the optimal control problem. The objective of the model for real-time operation model is to minimize the difference between the requested demand ($D_{req,\,z,t} Qb_z$) and the satisfied demand ($D_{sat,\,z,t} Qb_z$) for all the service areas z and simulation times t.

$$\text{Min Obj} = \sum_{t}^{t=T} \sum_{z}^{z=Z} \left[(D_{req,\,z,t} - D_{sat,z,t}) Qb_z \right]^b \theta_{z,t} \tag{7-1}$$

Where, Obj is the objective function; $D_{req,\,z,t}$ and $D_{sat,\,z,t}$ are the requested and satisfied demand multipliers respectively for service area z at time t; b is the power index for objective function; Z is the total number of service areas in the system; T is the total simulation time; Qb_z is the average demand (gpm) observed during 24-hour operations of the WDS for service area z and $\theta_{z,t}$ is a penalty multiplier for not satisfying demand for service area z at time t.

The constraints include two types: (a) those solved implicitly by the EPANET model and (b) those incorporated into the objective function as bound constraint violations.

1. The equations solved implicitly by EPANET for hydraulic and water quality analysis
2. The conservation of mass at each junction node.
3. The conservation of energy for each pipe m connecting nodes i and j.
4. The bounds on the water level of storage tanks.

5. Equations to define water quality analysis

Bound constraints are incorporated as bound violations into the objective function.

1. The nodal pressure head minimum and maximum bounds.
2. Bounds on pump operation times.
3. Switch constraint for turning on or off of pumps.
4. Power requirement constraint of the pumps.
5. Water quality constraint defining minimum and maximum bounds on concentrations.
6. Bounds on “satisfied” demand multipliers.

7.2.4. <u>Genetic Algorithm</u>

Genetic algorithms solve an unconstrained problem which is a reduced optimization problem consisting of the objective function with the bound constraints incorporated as penalty functions. In summary the unconstrained optimization problem uses EPANET as stated above to solve constraints defined as the hydraulic and water quality constraints, and the bound constraints are incorporated into objective function as penalty functions to define the reduced objective which is solved using the genetic algorithm. The interfacing of the genetic algorithm in MATLAB and the EPANET model is implemented using a MATLAB – EPANET toolkit developed by Elíades (2009).

Figure 7-1 illustrates the optimization – simulation procedure interfaced with resilience computations. The optimization-simulation model is provided with inputs including the EPANET network file and the power availability (Pow (available) $_{pump,t}$) as a function of time t. Optimization-simulation iterations are performed to provide the optimal pump controls and demands satisfied which are used in the resilience computations (see Figure 7-1).

The optimization-simulation model presented by Khatavkar and Mays (2018) considers supplied water demands at a service area level or a pressure zone level for the example presented herein but can also be considered a at the nodal level. The supplied water demands are decision variables because in the case of extreme emergencies the requested (or required demands) may not be deliverable because of electrical power shortages and/or limited water availability so the objective is to minimize the differences between what can be satisfied and what is requested or required, but to also satisfy pressures.

7.2.5. Real-time Operations Framework During Limited Electrical Power Generation

The aim of optimizing the operations of a water distribution system under limited availability of electrical power is an attempt to satisfy the requested levels of demand at various locations while also meeting pressure requirements of the system. The objective statement formulated here is to minimize the differences between the requested demand and the satisfied demands in all the service areas in the system.

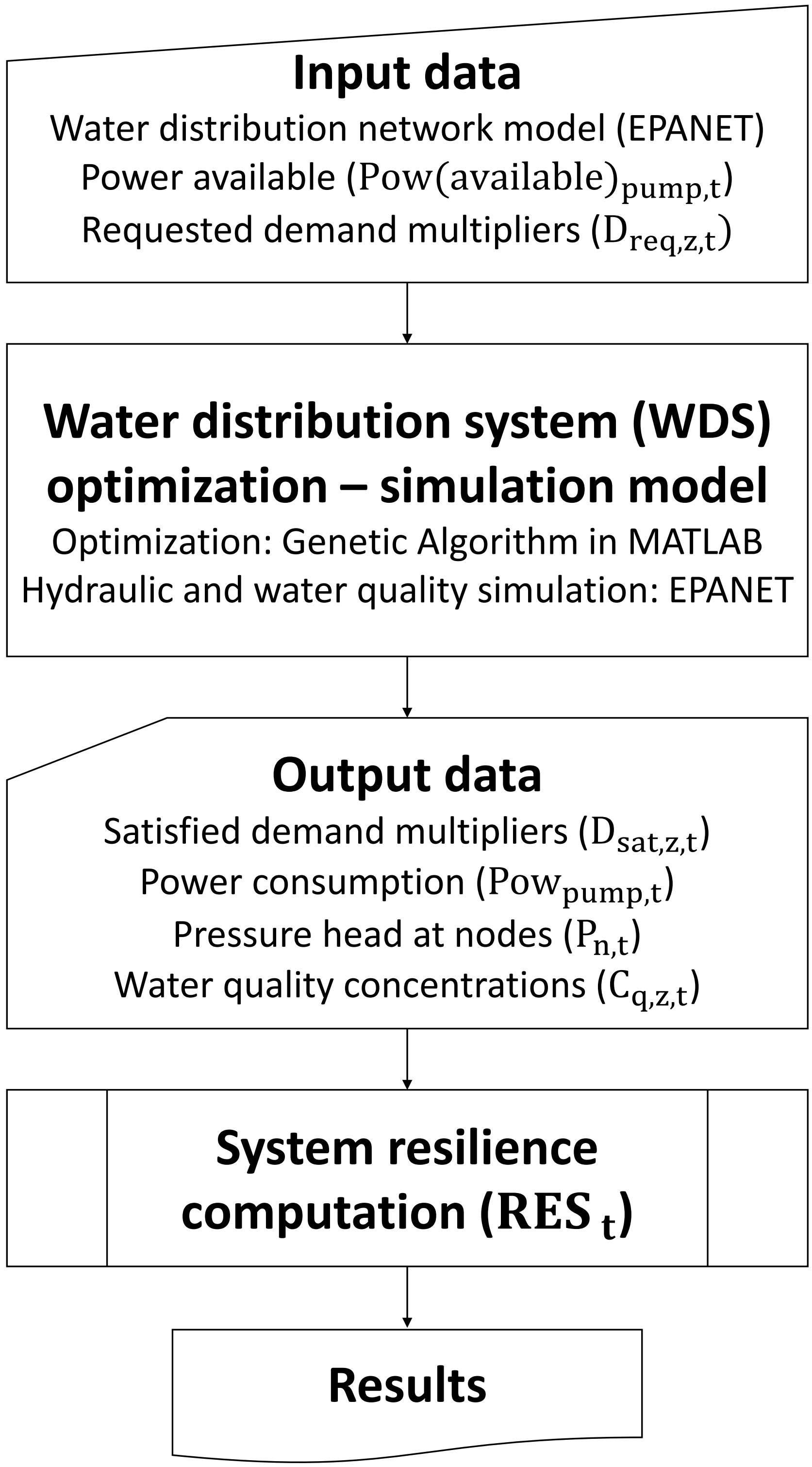


Figure 7-1 Resilience Computation Method Implementation

The following describes the overall concept of the real-time operation for each municipal water system:

8. At real-time t receive electrical power input from PDS.
9. Receive latest data (status of pumps, tank levels, status of valves, and flows in and out of the system from the supervisory control and data acquisition (SCADA) system for each water distribution system (WDS).
10. Update the EPANET for each water distribution system (WDS) input using data from the SCADA system for each WDS.
11. WDS optimization model is run to determine the actual demand pattern and pump operation that can be met with the limited electrical energy input. During the optimization/simulation process, the EPANET model is called repeatedly from the genetic algorithm optimization to determine WDS operations over the next 24 hours. The optimization model searches over the decision variables (pump operation and a satisfied demand pattern) to minimize the difference between demand requested and demand satisfied. The simulator determines the values of the state variables (nodal pressure heads, pipe flows, and tank levels) for each set of control variables (pump operations and satisfied demand multipliers) determined in the optimizer.
12. Implement the optimal pump schedule over the next hour which is accomplished through the SCADA system.
13. Repeat steps 1 through 5 each time increment during the emergency event each time incrementing the time $t = t + \tau$ with $\tau = 1$ hour.

In this approach feedback from the EPANET model is provided at each iteration of the overall real-time operation model. The EPANET output from this process includes satisfied demand multipliers ($D_{sat, z, tim}$), power consumption ($Pow_{pump, tim}$), pressure heads at nodes ($P_{n,tim}$) and water quality ($C_{q,n,tim}$) at simulation time tim. The system performance resilience (RES_t) is computed at real time (t) for simulation using the approach given in Figure 7-1.

7.3. Operational Resilience Computations

The overall objective is to define a resilience as a weighting of the pressure, power, demand and water quality satisfaction. A function in this study is defined as a measure of performance efficiency of the WDS with respect to a performance parameter (pressure, power, demand and water quality satisfaction).

The respective functions for pressure, power availability, demand satisfaction and water quality are defined as follows

7.3.1. Pressure Function

Pressure bounds are an important performance assessment criterion for water distribution system (WDS) operations. During emergency conditions, water demands may be satisfied at lower pressures than generally acceptable. Although lower pressure bound violations are a partial failure of the water distribution system since prolonged lower pressure bound violations could be a public health safety hazard. Similarly, water demands could be satisfied at pressures higher than the upper pressure bounds, which may lead to failure of WDS components or household faucets. Therefore, from an operational resilience perspective, pressure bound satisfaction is important for the demands that can be satisfied.

Pressures within a WDS are maintained within a range defined by lower and upper bounds for all nodes in the system. A low-pressure function $a\{P_{n,tim}\}$ is formulated, which has a value of 1 when node pressures are above $P_{s,low}$, while the function value is 0 for pressures lower than P_{low}. The pressure function is allocated a value (between 0 and 1) based on its deviation from the $P_{s,low}$ safe limit for pressures between P_{low} and $P_{s,low}$. The decision conditions for low pressure indication function for nodes 'n' and simulation time 'tim' are as follows:

If $P_{n,tim} \leq P_{low}$, then, $a\{P_{n,tim}\} = 0$

Else, if $P_{n,tim} > P_{low}$ and $P_{n,tim} \leq P_{s,low}$, then, $a\{P_{n,tim}\} = \min\left(0, \left(\frac{P_{n,tim} - P_{low}}{P_{s,low} - P_{low}}\right)\right)$

(7-2)

For pressures higher than the safe limit of $P_{s,upper}$, a high-pressure indication function $b\{P_{n,tim}\}$ is formulated, which has a value of 1 when node pressures are within the safe higher limit of $P_{s,upper}$, while the function value is 0 for pressures higher than the extreme limit of P_{upper}. The pressure function is allocated a value between (0 and 1) based on its deviation from the safe limit of $P_{s,upper}$, if the pressure lies within $P_{s,upper}$ and P_{upper}. The decision conditions for high pressure indication function ($b\{P_{n,tim}\}$) for nodes 'n' and simulation time 'tim' are as follows:

If $P_{n,tim} \geq P_{upper}$ then $b\{P_{n,tim}\} = 0$

Else, if $P_{n,tim} \geq P_{s,upper}$ and $P_{n,tim} < P_{upper}$.

$$\text{then } b\{P_{n,tim}\} = \max\left(0, \left(\frac{P_{upper} - P_{n,tim}}{P_{upper} - P_{s,upper}}\right)\right) \tag{7-3}$$

A combined function $f\{P_{n,t}\}$ for pressures is defined as follows

$$\text{If } a\{P_{n,tim}\} = 0 \text{ or } b\{P_{n,tim}\} = 0\,, \text{ then } f\{P_{n,tim}\} = 0$$

$$\text{Else, } f\{P_{n,tim}\} = \frac{a\{P_{n,tim}\} + b\{P_{n,tim}\}}{2} \tag{7-4}$$

Figure 7-2 illustrates the values of combined function $f\{P_{n,t}\}$ for the upper and lower pressure bounds.

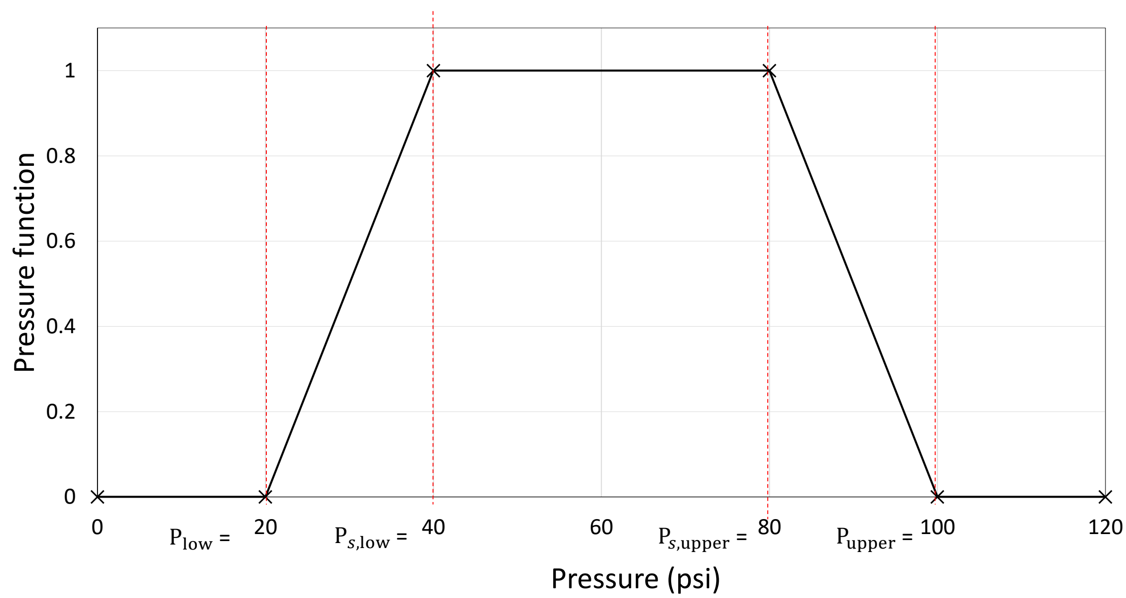


Figure 7-2 Pressure Function Bounds

7.3.2. <u>Power Availability Function</u>

An important function of the model for scheduling pump operations and demand satisfaction is to avoid power availability constraint violations. For an improved operation resilience compliance with the power availability as one of the major functions. The function for power, $f\{Pow_{pump,t}\}$ for all pumps and times t are considered to be binary (0,1). The function has a value of 1 for the pumps complying with the power availability, i.e the power consumed by the pump at that time is within the power available. The function assumes a value of 0 when the power availability is not satisfied. The decision rule for the power compliance function is as

$$\text{If } \mathrm{Pow}_{\mathrm{pump,t}} > \mathrm{Pow(available)}_{\mathrm{pump,t}} \text{ then } f\{\mathrm{Pow}_{\mathrm{pump,t}}\} = 0$$

$$\text{If } \mathrm{Pow}_{\mathrm{pump,t}} \leq \mathrm{Pow(available)}_{\mathrm{pump,t}} \text{ then } f\{\mathrm{Pow}_{\mathrm{pump,t}}\} = 1 \tag{7-5}$$

7.3.3. Demand Satisfaction Function

The function for demand satisfaction is given by $\frac{D_{sat,z,t}}{D_{req,z,t}}$, which is an output from the optimization model. The values show the extent to which the requested demands are satisfied. The function for demand satisfaction is given as

$$f\{\mathrm{demands}_{z,t}\} = \frac{D_{sat,z,t}}{D_{req,z,t}} \tag{7-6}$$

7.3.4. Water Quality Function

Meeting the end user water quality requirement is another important performance criterion for WDS operations. From an end user quality monitoring standpoint, residual chlorine for domestic users and TDS levels for power plants are parameters of importance. This study considers a generalized constraint for concentration ($C_{q,z,tim}$) of a water quality parameter (q) at simulation time t and water supply zone z. The concentration of such a substance/ parameter in the water supplied to the end user is generally governed by the water quality standards of the water district and the specific water supply requirements of the customer. Each parameter for water quality is bound by a lower and upper limit for concentrations given by $\underline{C}_q$ and $\overline{C}_q$ respectively.

$$\text{If } C_{q,z,tim} \leq \overline{C}_q \text{ and } C_{q,z,tim} \geq \underline{C}_q, f\{C_{q,z,tim}\} = 1$$

$$\text{Else } f\{C_{q,z,tim}\} = 0 \tag{7-7}$$

7.3.5. System Operation Resilience

Aydin, et al. (2014 a and b) used the following definition of resilience for functions of pressure, demand, and power in WDS during time t as

$$\mathrm{RES}_{f,t} = \frac{\text{\# of times satisfactory follows unsatisfactory}}{\text{\# of times unsatisfactory occurs}} \tag{7-8}$$

This study uses the definition of resilience shown above for deriving the equations for operational resilience under emergency condition.

7.3.6. Pressure Resilience

Resilience for the upper and lower pressure bound function $\left(f\{P_{n,tim}\}\right)$ defined in equation 7.4 is formulated using equation 7.8. The resilience for pressure bound satisfaction function is given as follows.

$$\mathrm{RES}_{pressure,t} = \left(\frac{\sum_{n}^{N}\sum_{tim}^{T}\max\left(0, f\{P_{n,tim}\} - f\{P_{n,tim-1}\}\right)}{\text{\# of times } f\{P_{n,tim}\}<1}\right)_t \tag{7-9}$$

Where $\mathrm{RES}_{pressure,t}$ is the resilience related to pressure functions at real-time t, $f\{P_{n,tim}\}$ is the function of pressure bound satisfaction defined in Equations 7.3 and 7.4 for simulation time 'tim' and node n.

7.3.7. Power Consumption Resilience

Resilience for power availability function $\left(f\{Pow_{pump,t}\}\right)$ defined in Equation 7.5 is formulated using Equation 7.8. The resilience for power availability function is given as follows.

$$\mathrm{RES}_{pow,t} = \left(\frac{\sum_{pump}^{PUMP}\sum_{tim}^{T}\max\left(0, f\{Pow_{pump,tim}\} - f\{Pow_{pump,tim-1}\}\right)}{\text{\# of times } f\{Pow_{pump,tim}\}=0}\right)_t \tag{7-10}$$

Where $\mathrm{RES}_{pow,t}$ is the resilience related to power functions at real-time t, $f\left(Pow_{pump,tim}\right)$ is the function of power availability defined in Equation 7.5 for simulation time 'tim' and pump.

7.3.8. Demand Satisfaction Resilience

Resilience for demand satisfaction function $\left(f\{demands_{z,t}\}\right)$ defined in Equation 7.6 is formulated using Equation 7.8. The resilience for demand satisfaction function is given as follows.

$$RES_{demands,t} = \left(\frac{\sum_{z}^{Z}\sum_{tim}^{T}\max\left(0,f\{demand_{z,tim}\}-f\{demand_{z,tim-1}\}\right)}{\#\text{ of times } f\{demand_{z,tim}\}<1}\right)_{t} \quad (7\text{-}11)$$

Where $RES_{demands,t}$ is the resilience related to demand satisfaction at real-time t, $f\{demands_{z,t}\}$ is the function of demand satisfaction at simulation time 'tim' and zone 'z'.

7.3.9. Water Quality Resilience

Resilience for water quality function $\left(f\{C_{q,z,tim}\}\right)$ defined in Equation 7.6 is formulated using Equation 7.8. The resilience for water quality function is given as follows.

$$RES_{WQ,t} = \left(\frac{\sum_{q}^{Q}\sum_{z}^{Z}\sum_{tim}^{T}\max\left(0,f\{C_{q,z,tim}\}-f\{C_{q,z,tim-1}\}\right)}{\#\text{ of times } f\{C_{q,z,tim}\}<1}\right)_{t} \quad (7\text{-}12)$$

Where $RES_{WQ,t}$ is the resilience for water quality at real-time t and $f\{C_{q,z,tim}\}$is the function of water quality at simulation time 'tim' and zone z.

7.3.10. System Resilience

The overall resilience is a weighting of the pressure, power, demand and water quality resiliencies

$$RES_{t} = \frac{W_{pressure}RES_{pressure,t}+W_{pow}RES_{pow,t}+W_{demands}RES_{demands,t}+W_{WQ}RES_{WQ,t}}{4} \quad (7\text{-}13)$$

where RES_{t} is the total resilience for real-time t; $W_{pressure}$, W_{pow}, $W_{demands}$ and W_{WQ} are weights associated with pressure, power, demand satisfaction and water quality resilience respectively. The values of these weights are determined through sensitivity analyses for a WDS based on importance of each parameter and its effects on the overall performance of the WDS. The values of these weights should be in compliance with the following equation.

$$W_{pressure} + W_{pow} + W_{demands} + W_{WQ} = 4 \tag{7-14}$$

7.4. Example Application for Operational Resilience

7.4.1. <u>Example WDS</u>

Two example WDS's are used to illustrate the application of the model. The first WDS system in Figure 7-3 does not include water storage tanks at the power plants, while the second system in considers a water storage tanks at each of the power plants in the system. Each example system includes two cities a reclaimed wastewater waste water treatment plant (WWTP), and a power distribution system (PDS). City 1 consists of four service areas (designated as nodes 1.1 – 1.4) with a total base demand of 30,000 gpm and City 2 (designated as nodes 2.1 – 2.5) consists of five service areas with a total base demand of 25,000 gpm. A total of 17 freshwater pumps and 11 reclaimed water pumps serve the WDS.

The power distribution system (PDS) is based on the IEEE 14 bus system (Kodsi and Canizares, 2003), which consists of five power plants. The cooling water for these power plants is supplied from both a freshwater source and the reclaimed water from a waste water treatment plant (WWTP)..

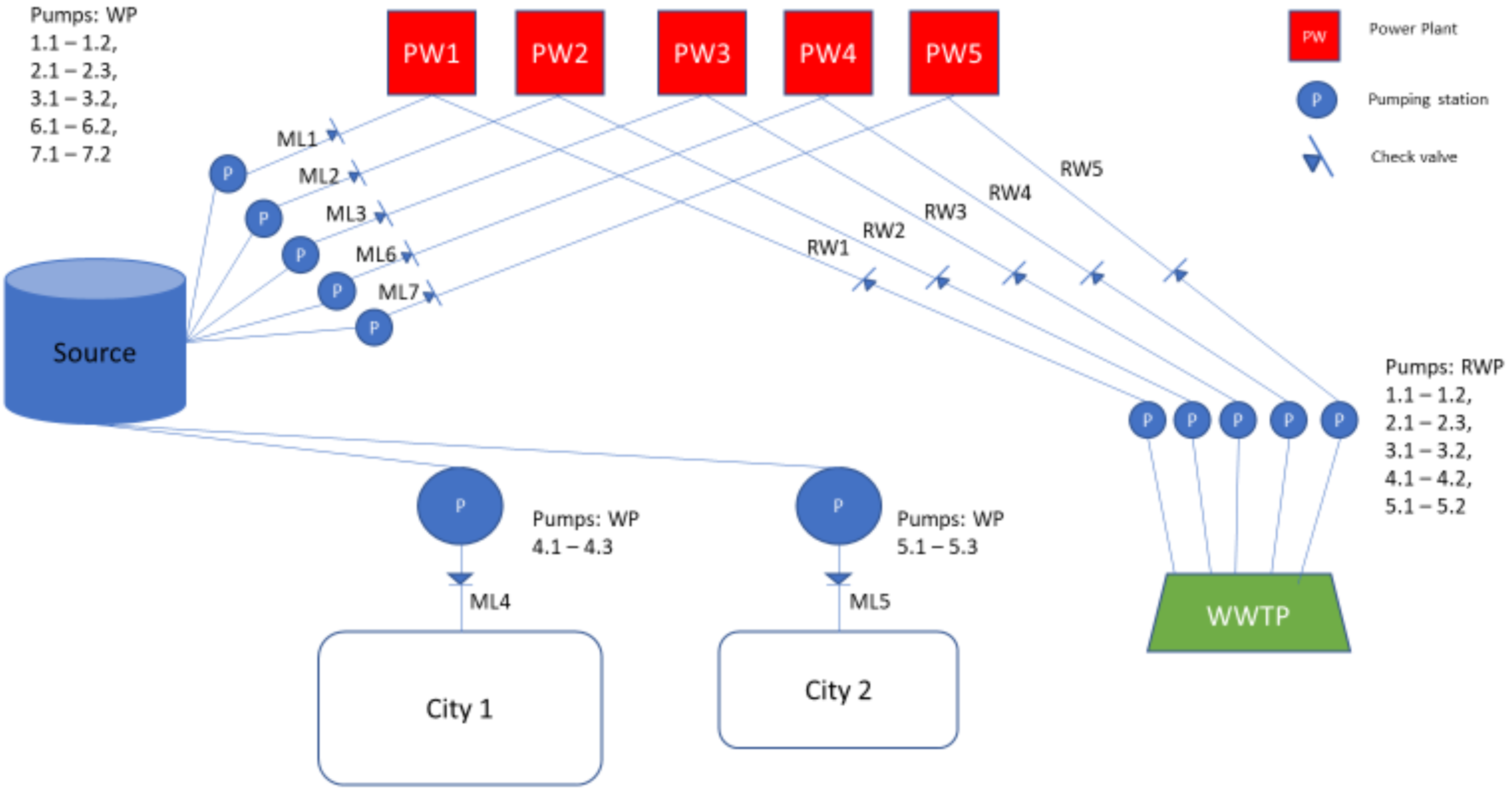


Figure 7-3 Schematic of Water Supply System WDS 1 (WDS 2 Includes Storage Tanks at Each of the Five Power Plants)

For the first WDS system (Figure 7-3), instantaneous mixing takes place for the fresh and reclaimed water supplied at the power plant node, whereas for the second WDS system the mixing takes place within the tank connected to the power plant Each service area has a separate demand pattern. The WDS has six demand patterns. One pattern applies to all the residential zones within the cities and for each of the five power plants in the system.

Pipes in the WDS are categorized into three types. Main lines (ML1 – ML7) which connect the freshwater pumps to the various power plants and cities; intermediate lines (IL 1.1 – IL 1.4 and IL 2.1 – IL 2.5) which are interconnecting nodes within the cities and reclaimed water pipelines (RW1 – RW5) which connect the waste water treatment plants to the five power plants in the system respectively. The pumps supporting the WDS are categorized as fresh water pumps (WP1 – WP7) and reclaimed water pumps (RWP1 – RWP5).

The optimization – simulation model (Khatavkar and Mays, 2018) was applied to the two example systems using the three scenarios of power outage in Table 7-1.

Table 7-1 Details of Power Outage Scenarios

Scenario #	Time of power outage (Hrs.)	Pumps affected	Consumers affected
1	8:00 to 10:00	WP 4.1, 4.2, 4.3, 2.1, 2.2, 2.3	City 1, Power Plant 1
2	6:00 to 12:00	WP 4.1, 4.2, 4.3, 2.1, 2.2, 2.3	City 1, Power Plant 1
3	8:00 to 10:00, 32:00 to 36:00	WP 4.1, 4.2, 4.3, 2.1, 2.2, 2.3 and RWP 2.1, 2.2, 2.3	City 1, Power Plant 1

The quality considerations for the three scenarios are

1. Scenario 1: No consideration of water quality

2. Scenario 2: Upper bound for total dissolved solids (TDS) of 2000 mg/l at power plant.
3. Scenario 3: Upper bound for total dissolved solids (TDS) of 1500 mg/l at power plant.

The operational resilience was applied to the WDS with storage tanks at each power plant for the three different levels of power shortage.

Pressures under normal conditions are between 40 psi and 80 psi, while the extreme limits are between 20 psi and 100 psi. The performance criteria for resilience is not achieved at all when the pressures cross the extreme bounds, while partial value is given to the function when the pressures are within the extreme bounds but cross the safe bounds. The pressure function (PF) is 100% when the pressures lie within the safe bound limits. The safe lower limit $P_{s,low}$ used for the example application in this paper was 40 psi and the lower limit P_{low} of 20 psi was used. The safe upper limit ($P_{s,upper}$) for pressures was 80 psi and an extreme upper limit (P_{upper}) of 100 psi. The combined function for pressures is shown in Figure 7-2.

Optimization-simulation model given by Khatavkar and Mays (2018) was applied in conjunction with the resilience computation methodology presented in this paper to the two WDS's discussed above for the three scenarios of short term power outages (Table 7-1). Optimization-simulation runs with resilience computations were also performed for varying degrees of long term limited power and water availability. The example applications were performed to assess the effects of water storage, short term power outages and long-term drought scenarios on the performance of water distribution systems and their ability to meet water demands.

The optimization model by Khatavkar and Mays (2018) is applicable both at service area level or pressure zone level or at the nodal level. We have applied the model to a network of over 1800 pipes. The example application used herein is a regional level system with each service area in the system represented as a node. WDS operators typically control the flow going into a particular service area using flow control valves (FCV) through the supervisory control and data acquisition (SCADA) system. For this study the WDS was skeletonized to the service area level to illustrate the level of controls available for state of art in practice. For a higher resolution implementation, separate demand patterns would have to be incorporated for each region. The level of skeletonization is dependent on the level or type of problem being solved.

7.4.2. Observations

The following observations are made based upon application of the optimization – simulation model and the resilience computation method (Figure 7-1). Figure 7-4 (a-c) show the system resilience for the scenarios 1, 2 and 3 applications to first example WDS 1.

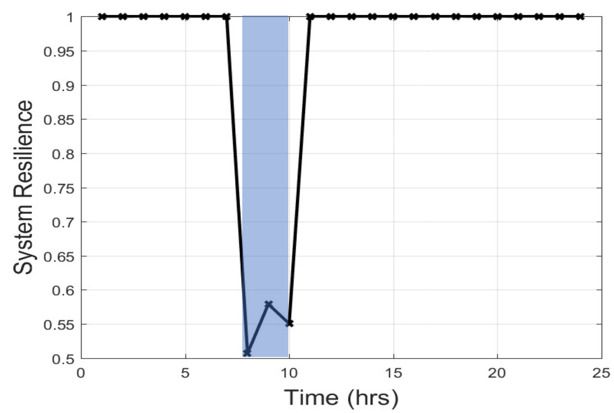
A decline in resilience is observed during the hours of limited power availability (see Figure 7-4 a) for scenario 1 application for the first example WDS. Similar observations were made during scenarios 2 and 3. Since no quality constraints are considered in this scenario 1, no changes in the resilience could be seen in the hours of normal power availability, as demands are satisfied during such times. The value of resilience (RES_t) is directly proportional to the demand satisfaction and inversely proportional to the number of violations of the power availability constraints and pressure bounds.

The system performance resilience of scenario 2 application for WDS 1 (see Figure 7-4 b) show a decrease in the resilience values is observed at hour 18. This decrease is attributed to a quality upper bound (2000 mg/l) imposed within the optimization model. It is evident in this scenario that the system resumed normal operations after the power shortage period.

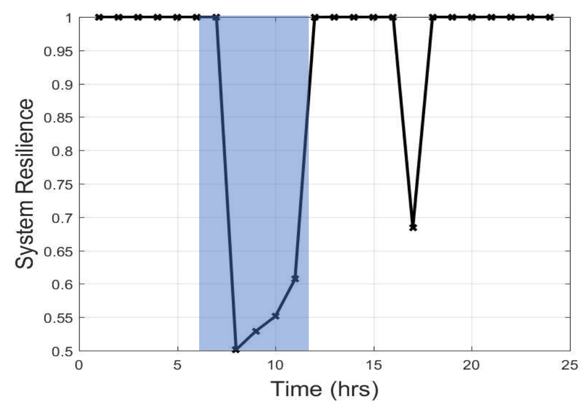
The resilience in scenario 3 (see Figure 7-4 c) show different trends as compared to scenario 2 (Figure 7-4 b), because of the more stringent water quality requirements (1500 mg/l) used for this application. The lower values of resilience observed in this case are due to the lower level of demand satisfaction at the power plants, due to quality constraints.

For the second WDS, the storage tanks provide the system with a redundancy in terms of stored water, which can be supplied in case of a contingency. The tank also provides a better control over the quality of water flowing into the power plants, since the mixing within the tank is not instantaneous and could be anticipated through modeling the fresh and reclaimed water flows going to the tank. With this increased robustness for the system, the tanks also provide for an increased complexity in terms of increased connections, components and costs. Figure 7-5 (a) illustrates the system resilience for scenario 1 application for WDS 2.

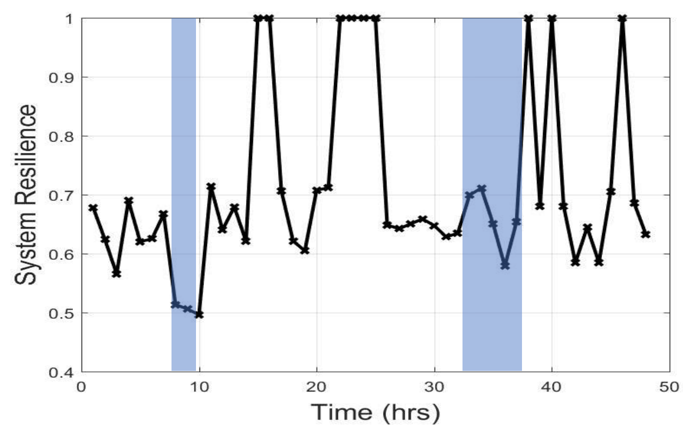
The results of resilience computation show that the system could fulfill its functions and did not suffer from any failure, even with a limited power availability of two hours at some pumps in the system (given in Table 7-1). Figure 7-5 (b) shows system resilience for scenario 2 application for second example WDS.



(a) Scenario 1

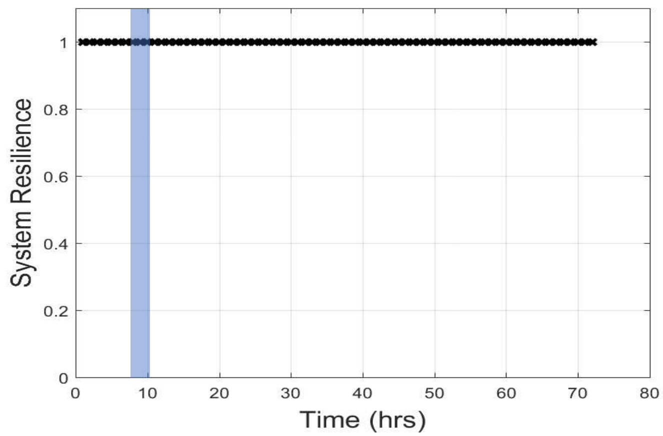


(b) Scenario 2

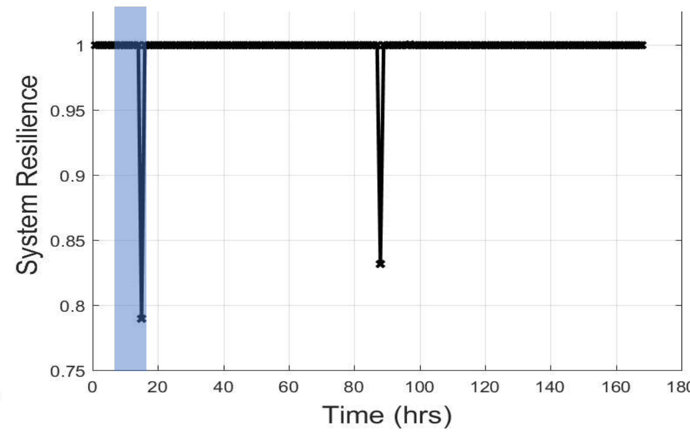


(c) Scenario 3

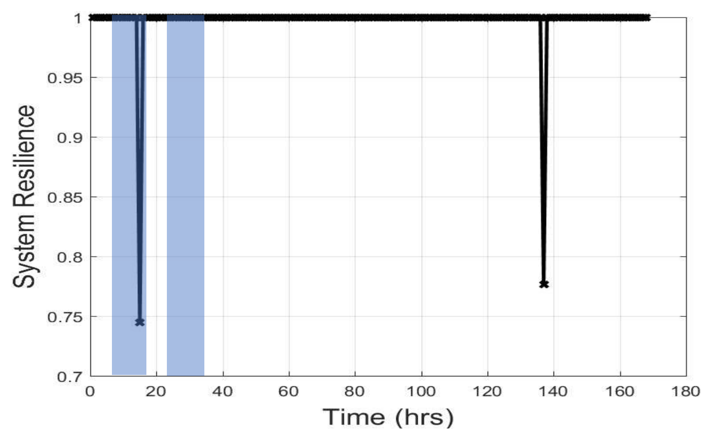
Figure 7-4 Operational System Resilience for WDS



(a) Scenario 1



(b) Scenario 2



(c) Scenario 3

Figure 7-5 Operational System Resilience for WDS 2

It is also evident that since system resilience values are based on recovery of the WDS from a failure, lower values are only observed after a failure occurs and not because of any other characteristics of system performance. Lowest system resilience values are observed during the hours of limited power availability only for a short period, rather than failing in fulfilling the functions for the entire emergency period.

The second example WDS shows an improved performance because of the increased functionality provided by the storage tanks in the system. The scenario also consists of a TDS upper bound of 2000 mg/l for the power plant nodes, which were observed to be adhered to for the entire application. This shows an increased fulfilment of the system functions for limited power availability conditions as well as normal operating conditions. Resilience for scenario 3 application for WDS 2 are illustrated in Figure 7-5 (c). The results are similar to Figure 7-5 (b), which illustrates that even the stringent quality upper bounds (1500 mg/l) do not affect the resilience of the system for this WDS, unlike the results for the same scenario observed in Figure 7-4 (c) for first system.

Application of the model for a long-term drought scenario, was modelled with different levels of water and electrical energy supply shortages. Figure 7-6 shows the system resilience for 24-hour simulation/optimization under different conditions of water availability. The demands were satisfied to the requested levels to about 55000 gpm of water availability and thereafter reduced with the reducing water availability. The system resilience shows a decline with a decrease in the water available.

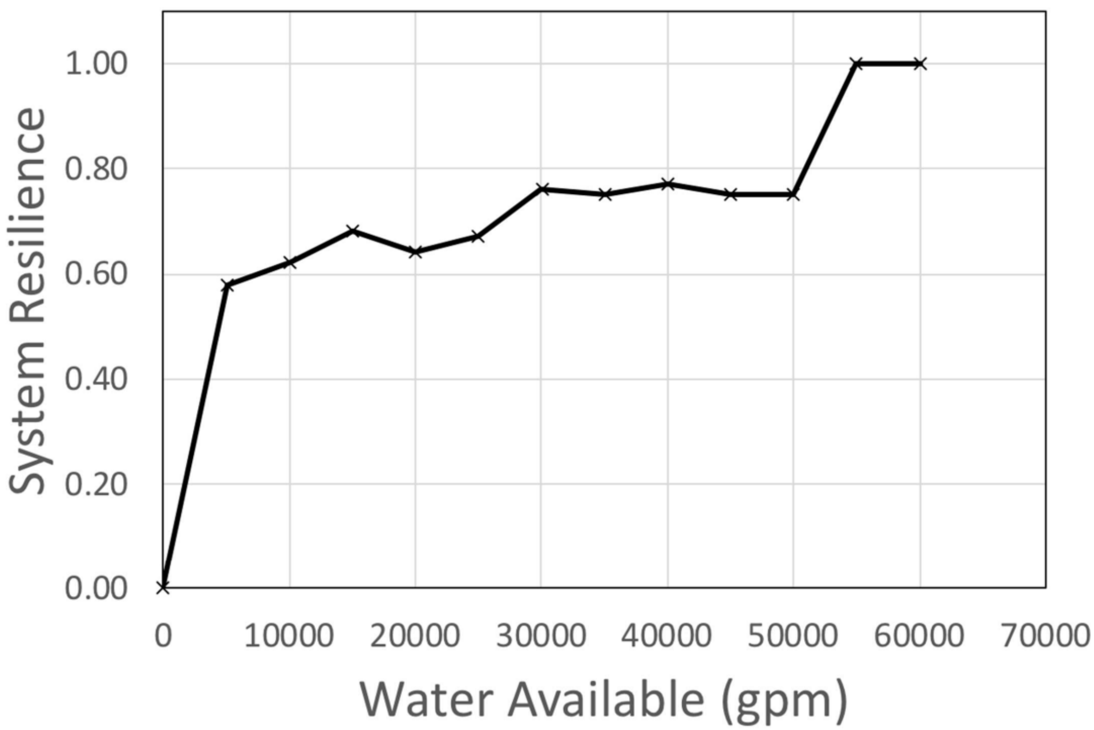


Figure 7-6 Operational System Resilience for Water Availability Conditions

System resilience for 24-hour simulation/optimization under different conditions of electrical energy availability are illustrated in Figure 7-7. For energy availability of up to 150000 kW-hr per day, it was observed that all the required system functions were achieved to the fullest and hence the resilience values show 100 percent values for all the values beyond that level of energy availability. With a long-term reduction in power availability of 100000 kW-hr, system resilience is between 0.75 and 0.8. This is followed by small decreases in resilience values until an energy availability of 40000 kW-hr is reached, where a system resilience of 0.7 is observed. The values of resilience show a steep fall for energy availabilities lower than 30000 kW-hr, since the pumps cannot be operated under lower energy availabilities.

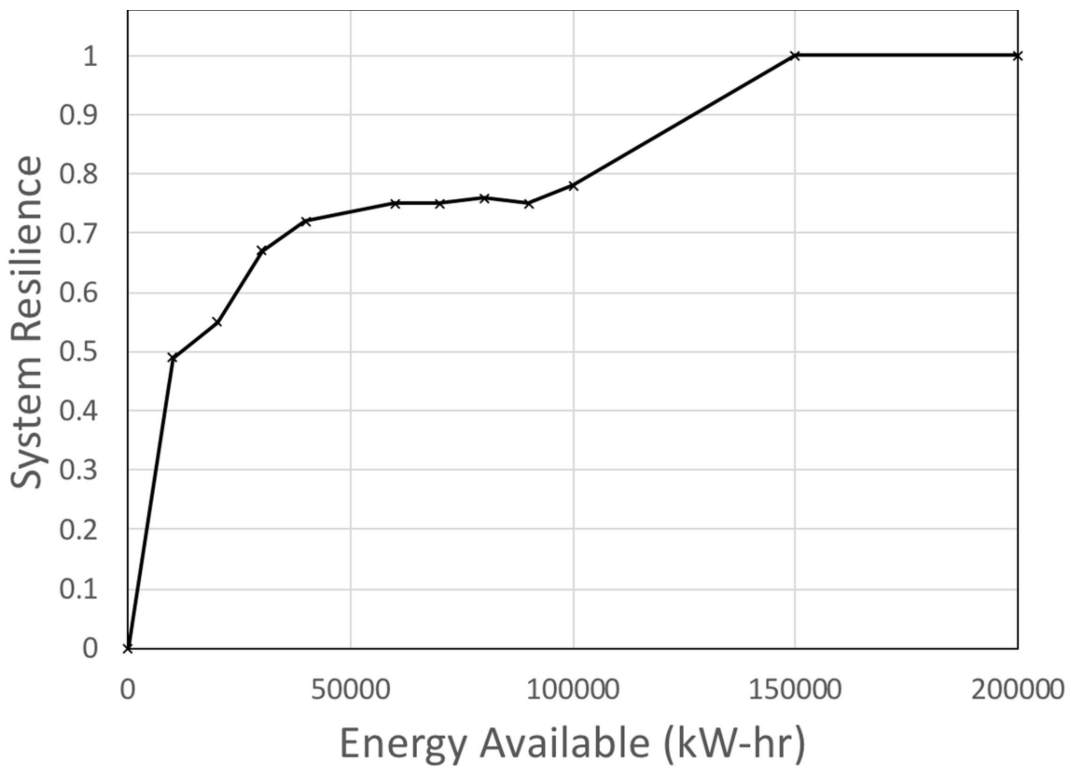


Figure 7-7 Operational System Resilience for Energy Availability Conditions

7.5. Infrastructural Robustness Metrics

Robustness is the inherent capability of a system to resist failures. This capability is an intrinsic property of a system based on its structure and configuration. This study considers metrics for computation of the system level robustness based on system structural configuration. The robustness metrics for the WDS are defined in the following sections.

7.5.1. WDS: Connectivity Metric

Connectivity of a node in a WDS is defined as the number of links (l_n) which are connected to a node n and characterizes the system interconnection and the degree of redundancy of the system components. The connectivity metric is given as

$$M_{Conn,n} = \mathrm{card}(l_n) \tag{7-15}$$

Where, $M_{Conn,n}$ is the number of links connected to a node n and l_n is the set of links (including pipes, pumps and valves) connected to node n.

7.5.2. WDS: Betweenness Metric

The betweenness metric gives the robustness associated with the functionality of the links connected to a certain node. This metric is based on the type (pipe, pump or valve) and size (diameter for pipe/valves or flow for pumps) of the connecting links. The betweenness metric scores the nodes based on the functional importance of the connecting links for WDS operations. A functionality factor (α_l) for link l is defined as a function of the link's type and size. For pipes α_l is based on the size of the pipe and is calculated as the ratio of the diameter of pipe l divided by the maximum diameter of all L pipes in the WDS. For pumps, α_l is based on the maximum possible flow from the pump and given as the ratio of the flow of a given pump divided by the maximum flow in any of the WDS pumps. The betweenness metric for a node n in a WDS is derived as

$$M_{bet,n} = \sum_{l(n)} \alpha_l \tag{7-16}$$

Where, $M_{bet,n}$ is the betweenness metric for node n.

7.5.3. WDS: Demand Priority Metric

Demand priority is an important robustness parameter during extreme water shortage and limited electric power availability conditions. A demand pattern is defined for each service area in a WDS. The demand priority metric is simply a priority weight given to each service area z to ensure robustness with respect to demand satisfaction at higher priority service areas. The demand priority metric ($M_{demand,\, z}$) values are assigned a non-integer value between 0 and 1 for each service area in the system depending on the type of demands (residential, commercial, industrial or cooling water for power plants) in the service area.

7.5.4. WDS: Demand Adjustment Metric

Satisfaction of water demands in a WDS is an important requirement during normal and emergency operating conditions. For extreme drought and electrical outage scenarios, a part of the water demand at certain service areas may not be satisfied. In such conditions, the willingness of the consumers to reduce the water consumption helps in operating the WDS to supply the required water to priority locations such as hospitals, public utilities or fire stations. The demand adjustment metric $\left(M_{demAdj,\, z}\right)$ gives the extent to which the population in a service area is willing to adjust their demands in an event of short and long-term water and electric power shortage. This metric is derived from extensive surveys conducted in Arizona for single family units and multi-family units. For every service area z in the system, two metrics are defined for single family units ($dAdj_{sf,t}$) and multi-family units ($dAdj_{mf,t}$) based on the proportion of single family units and multi-family units present.

7.6. Infrastructural-Operational Resilience Computations

7.6.1. WDS: Pressure IOR

The pressure IOR considering infrastructural robustness metrics for connectivity (equation 7.15) and betweenness (equation 7.16) at time t is given as follows

$$R_{f\{P_{n,tim}\},t} = \left(\frac{\sum_{n}^{N} \sum_{tim}^{T} \left[\left(\frac{M_{Conn,n}}{\sum_{n}^{N} M_{Conn,n}} \right) \left(\frac{M_{bet,n}}{\sum_{n}^{N} M_{bet,n}} \right) \max\left(0, f\{P_{n,tim}\} - f\{P_{n,tim-1}\}\right) \right]}{\#\text{ of times } f\{P_{n,tim}\} < 1} \right)_t \tag{7-17}$$

Where $R_{f\{P_{n,tim}\},t}$ is the infrastructural-operational pressure resilience at time t; $f\{P_{n,tim}\}$ is the performance evaluation function for pressure bounds for node n and simulation time tim.

7.6.2. WDS: Power Consumption IOR

The functionality factor for pumps (α_{pump}) is considered for computing the IOR for power consumption at time t as follows

$$R_{f\{Pow_{pump,tim}\},t}=\left(\frac{\sum_{pump}^{PUMP}\sum_{tim}^{T}\left[\left(\frac{\alpha_{pump}}{\sum_{pump}^{PUMP}\alpha_{pump}}\right)\max\left(0,f\{Pow_{pump,tim}\}\text{-}f\{Pow_{pump,tim-1}\}\right)\right]}{\#\text{ of times }f\{Pow_{pump,t}\}=0}\right)_t \tag{7-18}$$

Where, α_{pump} is the functionality factor for pump; $R_{f\{Pow_{pump,tim}\},t}$ is the power consumption infrastructural – operational resilience at time t; $f\{Pow_{pump,tim}\}$ is the performance evaluation function for power availability.

7.6.3. WDS: Demand Satisfaction IOR

Demand priority and demand adjustment metrics are considered for computation of the demand satisfaction IOR. A mathematical function $g\{demand_{z,t}\}$ is used to determine whether the satisfied demand (D_{sat}) is within the demand adjustment metric for a certain zone. The mathematical function is given as

$$g\{demand_{z,t}\}=\max\left(0,\frac{D_{sat,z,t}}{D_{req,z,t}}\text{-}mfn_z dAdj_{mf,t}\text{-}sfn_z dAdj_{sf,t}\right) \tag{7-19}$$

Where mfn_z and sfn_z are ratios of multi-family units and single-family units in service area z.

The computation for demand satisfaction resilience is as follows

If $g\{demand_{z,t}\}=0$, then $ds_{z,t}=0$

If $g\{demand_{z,t}\}>0$, then $ds_{z,t}=\frac{D_{sat,z,t}}{D_{req,z,t}}$ (7-20)

The infrastructural-operational resilience for WDS demand satisfaction is given as

$$R_{M_{demand,\,z},t}=\sum_{z}^{Z}\left(\frac{M_{demand,\,z}}{\sum_{z}^{Z} M_{demand,\,z}}\right) ds_{z,t} \tag{7-21}$$

7.6.4. <u>WDS: System IOR</u>

The total system infrastructural-operational resilience is computed as a weighted sum average of the infrastructural-operational resiliencies for pressure, power consumption and demand satisfaction as follows

$$R_{WDS,TOT}(t)=\frac{w_p\, R_{f\{P_{n,tim}\},t}+w_{pow}R_{f\{Pow_{pump,tim}\},t}+w_{dem}\, R_{M_{demand,\,z},t}}{w_p+w_{pow}+w_{dem}} \tag{7-22}$$

7.7. Example Application for WDS IOR

7.7.1. <u>Example System</u>

The example water system is a hypothetical WDS modeling demands from two cities and a PDS consisting of five power plants. The cooling water for the power plants is supplied from both a freshwater source and a reclaimed wastewater source (shown in Figure 7-3). City 1 has four service areas with a total base demand of 30,000 gpm. City 2 has five service areas with a total base demand of 25,000 gpm. A total of 17 freshwater pumps (WP) and 11 reclaimed water pumps (RWP) serve the overall WDS. Cooling water demands for power plants, on an hourly basis, are input to the WDS optimization – simulation model from the PDS optimization – simulation model. Each power plant in the system is equipped with onsite water storage equivalent to a 2-week average water consumption.

7.7.2. Application Scenario

The scenario considered for the example application has a combination of water and power contingencies. The water contingency experienced by the water system is representative of drought conditions that would result in limited water availability for extended periods of time (4 weeks). Figure 7-8 shows the trends in the system level water availability. The power contingency under consideration consists of several extended periods (black sky events) where two pumping stations (freshwater pumping station at power plant 1 and reclaimed water pumping station for power plant 2) experienced an outage. Table 7-2 contains a summary of the pumps that experienced the power contingency while Figure 7-9 shows the hours during which the power outages at pumping stations occurred.

The optimization-simulation model for WDS controls explained in chapter 4 was used for the example application of 4 weeks (672 hours).

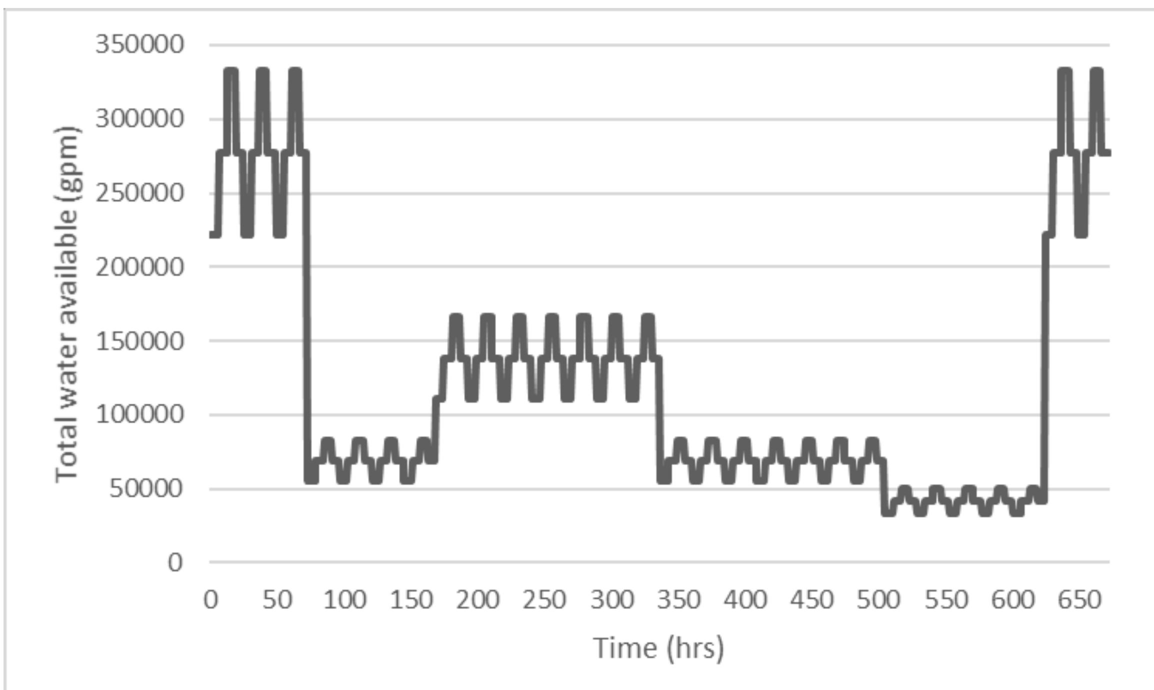


Figure 7-8 Trends of Water Availability

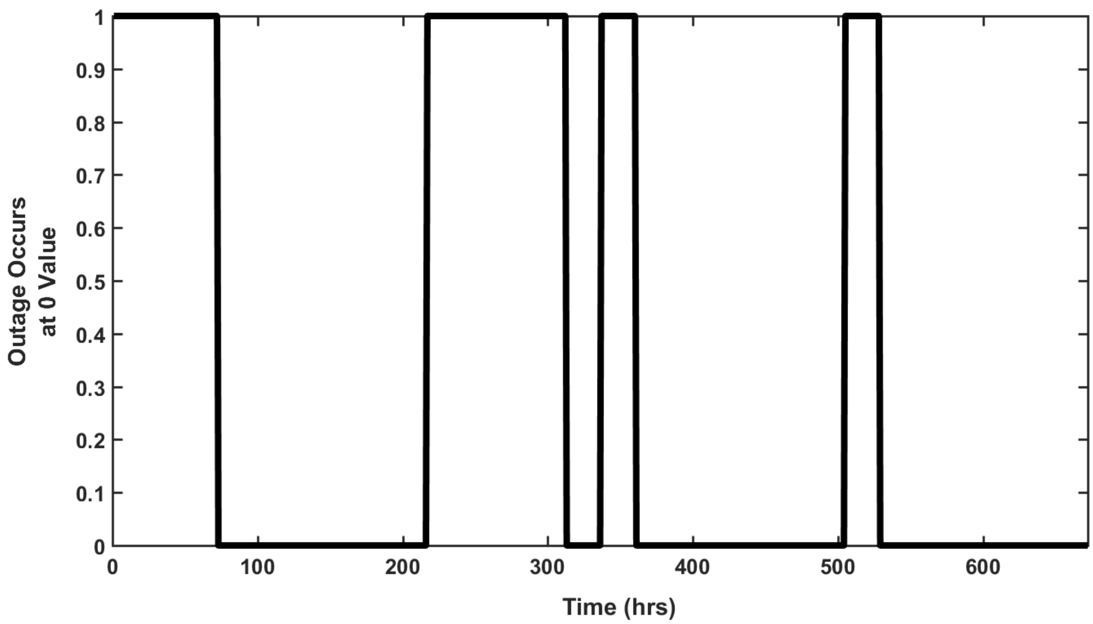


Figure 7-9 Pump Outage Hours

Table 7-2 Pumps Affected by Power Outage

	Pump Name	WDS Node	EPS Bus
1	RWP2.1	1	22
2	RWP2.2	2	23
3	RWP2.3	3	24
4	WP1.1	6	36
5	WP1.2	8	37

7.7.3. Results

Figure 7-10 shows the trends of WDS IOR computed using equation 7.22. The shaded areas in Figure 7-10 show the contingency time periods due to power outage or limited water availability. A moving average (average window = 24 hours) is used for smoothening the IOR data. WDS IOR is observed to decline from simulation times slightly before the contingency start time of 72 hours due to the data processing of the IOR used to post-process the results. The WDS is observed to recover from the reduced

IOR values at the end of the simulation period even though pump power outages still persist. This is because during the last 48 hours of the simulation there is no shortage of available water. Thus, it is noted that the effect of the pump power outages is mitigated in the WDS as the simulation progresses. It is also noted that the effect of the second round of severe contingencies beginning in the 336th hour on the IOR values is not as severe as the effect of the first contingency that occurs in hour 72, demonstrating that the implemented optimization-simulation strategies effectively attenuate the impact of the second contingency conditions prior to hour 336.

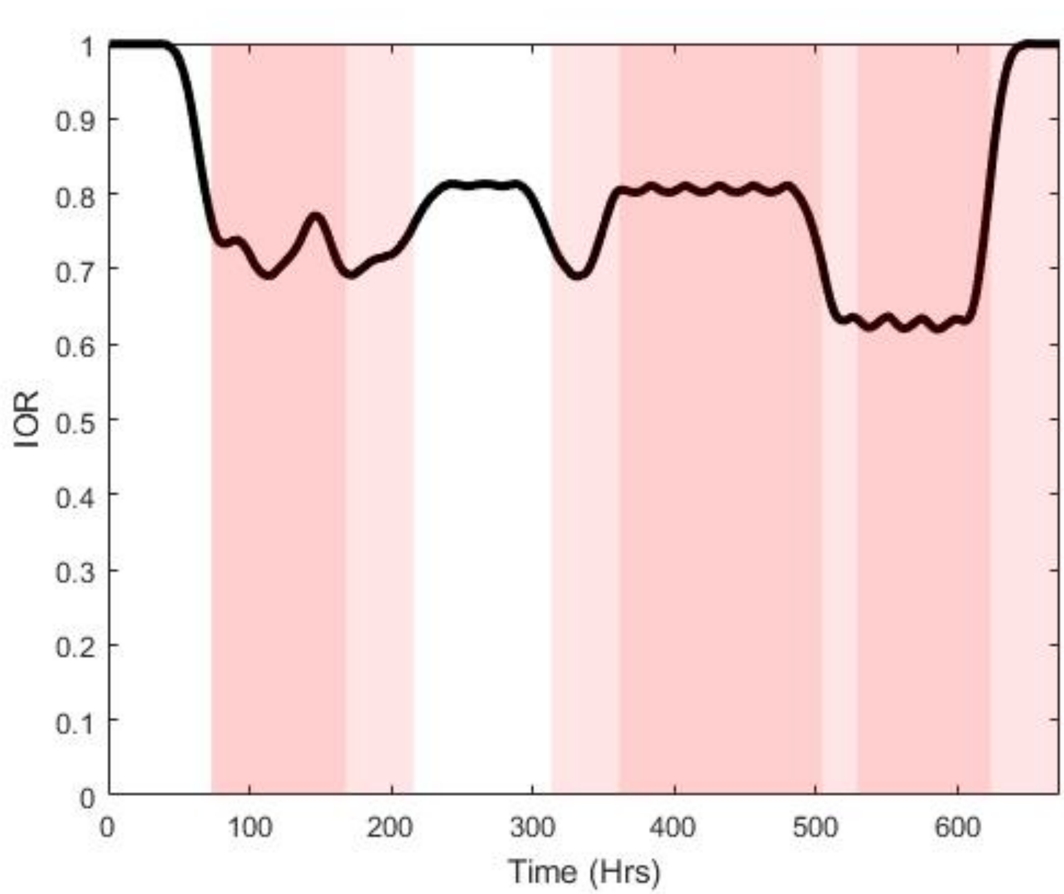


Figure 7-10 Infrastructural – Operational Resilience (IOR) for WDS Operations

7.8. Conclusions

Performance of water distribution systems under the conditions of limited power and water availability is an important consideration for design and operation of a water smart city. The aim of this study is to present a mathematical formulation for system performance resilience under considerations of limited water and power availability conditions.

Resilience computation method presented in this paper provides for a real-time WDS performance assessment tool. The interactive nature of the optimization-simulation method used in conjunction with the resilience computation methodology would provide the WDS utility a real-time performance analysis and decision-making tool. In addition to the real-time operations applications, the methodology presented in this study could also be applied to anticipatory drought scenario modeling and policy making. Decision making using the resilience computation methodology would involve several scenario and risk analyses considering the various forecasted climate conditions. This paper shows the foundation for mathematical formulation of an overall anticipatory drought management tool based on WDS performance analysis and WDS control optimization for water-energy nexus considerations.

The operational resilience computation methodology (shown in Figure 7-1) was applied to two regional level example WDS's coupled with a supporting PDS. The system resilience was computed based on four criterion including demand satisfaction, pressure bound compliance, power availability and water quality considerations. Results of the application of resilience computation methodology for the example systems show the applicability of the model for short term and long term limited power availability and for drought scenarios. Water storage tanks at the power plants in the second WDS improve the overall operational resilience as compared to the first WDS.

The infrastructural – operational resilience computational method presented in this paper provides for the inclusion of infrastructural robustness calculations within an extended time-period WDS optimization-simulation framework (explained in chapter 4). The IOR computation methodology was applied to a regional level WDS coupled with a corresponding EPS. Results of the application of this computation methodology for the example systems show the ability to capture system resilience when modeling short term and long term electric power outages and drought scenarios. This optimization-simulation framework used in conjunction with the methodology for IOR computation would allow utilities the ability to analyze real-time performance and facilitate decision-making. In addition, the IOR methodology could be applied to forecasted, extreme, mega-drought scenarios for more accurate system simulations and anticipatory policy making. The IOR computation methodology presented in this paper does not consider component failures in WDS in the computation procedure.

Future work will involve application of the developed IOR computation methodology to larger WDS and EPS test systems. Moving the application towards larger, more realistic test systems also warrants the inclusions of a developed middleware architecture, which emulates the SCADA communication that would be necessary for data exchange between the two networks.

8. REAL-TIME OPERATION OF WATER SUPPLY CANAL SYSTEMS UNDER LIMITED ELECTRICAL POWER AND/OR WATER AVAILABILITY

8.1. Summary

A new framework is presented for the real-time operation of water supply canal systems (WSCS) under critical conditions during short term and long-term emergency events such as limited electrical energy and/or limited water availability, electrical grid failures, extreme droughts or other severe conditions related to natural and manmade disasters. WSCS are used for conveyance of raw-water from sources such as lakes, reservoirs or rivers to water treatment plants which supply treated water to consumers through water distribution systems (WDS). The approach interfaces an optimization -simulation model for WSCS with an optimization -simulation model for WDS to provide for a comprehensive decision-making tool for the control of WSCS and WDS. Two WSCS optimization methodologies are presented including a non-linear programming approach and an optimization-simulation approach that interfaces a genetic algorithm (MATLAB) with the U.S. Army Corps of Engineers HEC-RAS simulation model. Steady state analysis of the WSCS is performed for each time period of operation. The new methodologies for determining pump and gate operations under limited power and/or water availability are illustrated using two example canal systems.

8.2. Introduction

The major dependency of electric power systems (EPS) on water delivery systems, including both water distributions systems (WDS) and water supply canal systems (WSCS) is the requirement of water for the cooling cycle of thermoelectric power generation. The main dependency of the water system on the electric system is the electrical power required for pumping water from the sources to the treatment plants and then to users via the WDS (pressurized pipes) and/or WSCS (open channels). Water usage for power plants includes both water withdrawal and water consumption. The withdrawal and consumption rates of modern power plants are dependent on many factors such as open or closed cooling cycles, the types of equipped emission control schemes and the power plant's location. The consumption of water in power plants is due to evaporation and drift losses associated with the type and design of the cooling tower, and the required blowdown which is a design parameter dependent on water-quality.

Canals are used to supply water for agriculture, municipal and industrial use, fish and wildlife enhancement, decreasing flood damage and power generation (Buyalski et al., 1991). WSCS can be large consumers of energy for pumping especially at locations where the topography of the region mandates lifting of water. For example, the Central Arizona Project (CAP) is the largest single consumer of electricity in Arizona (Lamberton et al., 2010; Scott et al., 2011; Eden et al., 2011). CAP uses approximately 2.8 million megawatt-hours of energy (about 4 percent of all the energy consumed in Arizona) to deliver about 1.5 million acre-feet of water to central and southern Arizona (Lamberton et al., 2010). Pumping plants lift water 2900 feet over the length of the CAP canal. Scott et al. (2011) documented in detail the water-energy nexus policy implications for canal systems in general and CAP in particular. Cost of energy required for pumping is about 80% of the total cost incurred for urban water supply in the United States (Lamberton et al., 2010). The reliability of canal operations during natural and manmade emergency conditions is an important factor while considering the interdependencies between the water and energy systems.

Figure 8-1 illustrates a communication overlay implemented via a middleware architecture for real-time operation of EPS, WDS and WSCS based upon the exchange of real-time data. Software defined networking (SDN) can be used to represent the communication overlay implemented via a middleware architecture (see Figure 8-1). This overlay enables a reliable and efficient data exchange between the three (EPS, WDS and WSCS) otherwise isolated supervisory control and data acquisition (SCADA) systems. SCADA systems are used on most water supply canal systems (WSCS) around the world (Clemmens, 2006). Provision of SCADA systems for remote monitoring and control of WSCS allow for automation and optimization of WSCS operations. One of the major challenges in water supply systems management is better control for flow through every component of the system. Automation of canal controls is comprised of centralized remote - control systems through a communication system and a control mechanism. Mareels et al. (2005) explained the various patented local and remote canal controllers used for automation of entire canal systems to meet water orders in real-time.

A new methodology has been presented by Khatavkar and Mays (2018) for the short-term or long-term real-time operation of water distribution systems (WDS) under critical conditions considering the quantity and quality requirements of the various water demands. The approach by Khatavkar and Mays (2018) interfaces an optimization procedure (genetic algorithm) with the US Environmental Protection Agency's EPANET simulator in the framework of an optimal control problem in MATLAB.

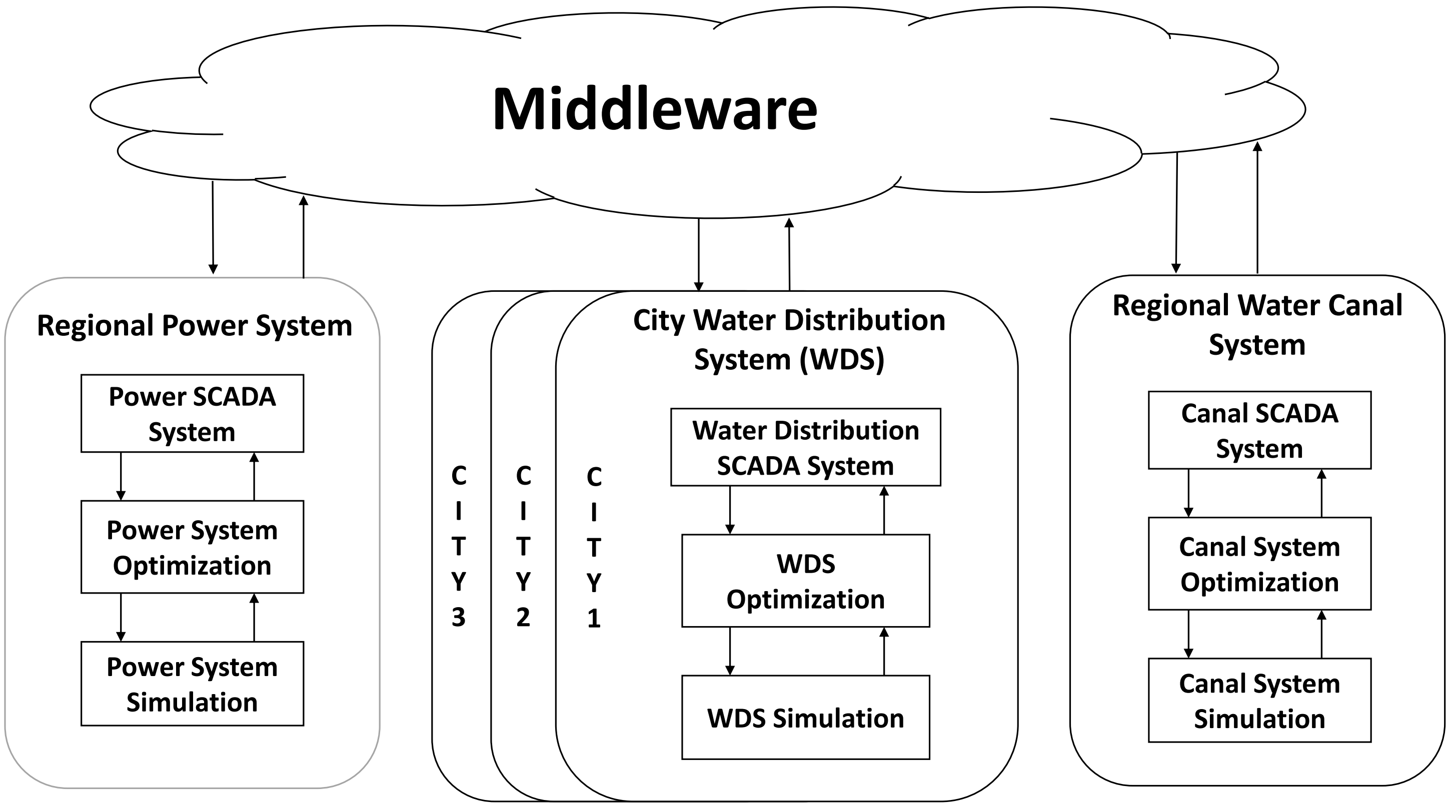


Figure 8-1 Communication Overlay Implemented via a Middleware Architecture for Real-time Operation Model

The basic objective of optimizing the operations of a WDS under limited availability of energy and/or water is to minimize the difference between requested demands and those that can be satisfied while meeting pressure and water quality requirements of the system.

A new methodology is presented herein for the real-time operation of WSCS under critical conditions that can be used in conjunction with the optimization-simulation model by Khatavkar and Mays (2018).

8.3. Water Supply Canal Systems

A canal system can consist of a single canal or a more complex system with several diverging canals that deliver water to various demand locations. An example dendritic WSCS (shown in Figure 8-2) consists of a primary canal and sub-canals. Water supply canal systems (WSCS) are manmade channels for water conveyance engineered to provide a controlled flow of water. A regulating structure in an open channel system is used to regulate the flow passing through weirs, control inlets, stop logs and slide gates (Aisenbrey et al., 1978). Along the main channel, there can be several check structures, that control the level of water in the canal. These check structures can also have turnouts (or offtakes) on their upstream, from which water is diverted either to a sub-canal or directly to the user. The offtakes could either be gravity offtakes (with no lifting and gravity flow) or lift offtakes (where pumping is required for lifting water from the main canal). Check structures, which are usually provided with gated outlets, are required to maintain water levels at the offtakes. Large scale pumping is required in canal systems where the topography of the region mandates lifting of water over large elevations.

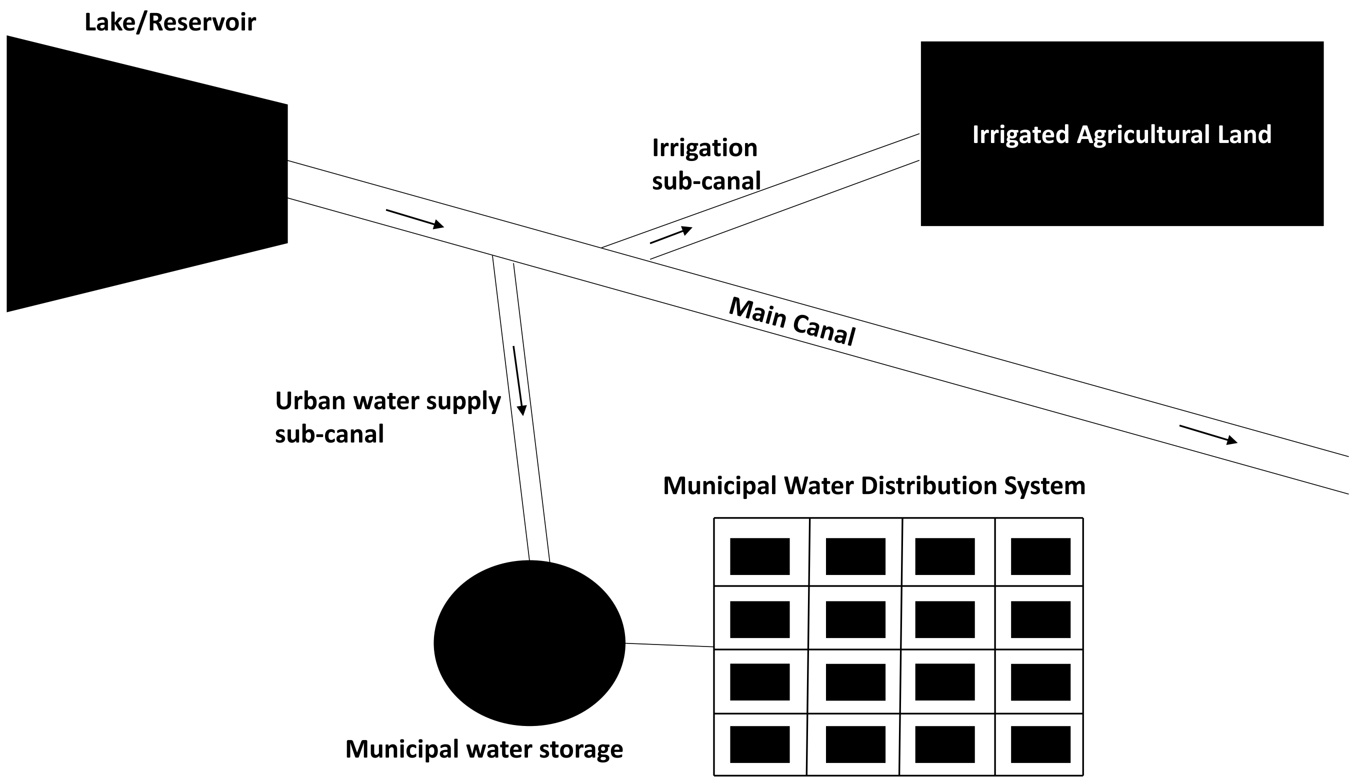


Figure 8-2 Schematic of a WSCS

The single canal illustrated in Figure 8-3, is comprised of several reaches defined by cross-sections j = 1, 2, …, J. At some cross sections water is diverted from the canal to provide water to users. The canal can have control structures such as gates in addition to pumping stations. As shown in Figure 8-3, a reach is used to define a pumping station. A pumping station can have P pumps working in series. Pumps are used in canal systems for lifting water out of the canal (lateral flow boundary) or at locations where a head difference warrants lifting of water from an upstream point in a canal with an elevation lower than the downstream point.

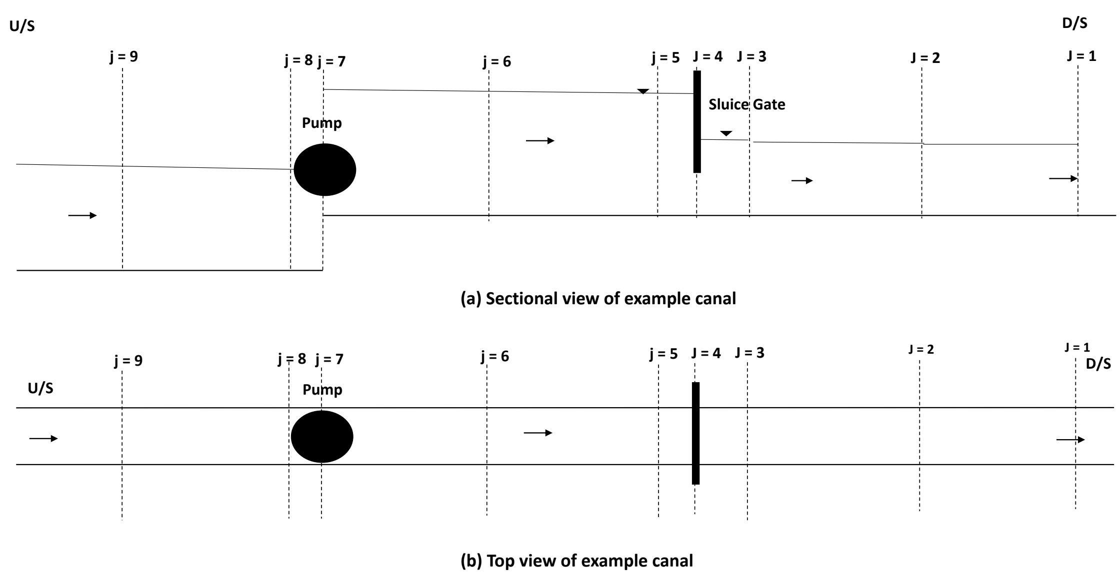


Figure 8-3 Example Canal System 1 Showing Cross-section Location, Pumping Stations and Gate Structures

WSCS typically have variable speed pumps to achieve demand-based control and energy savings. Variable speed pumps automatically adjust their revolutions or speed via a regulating loop. Varying the speed of the pump creates the same effect as installing a different-size impeller on the pump resulting in a different pump curve every time the speed is changed. The methodology considers pump speeds and gate controls as decision variables, on an hourly basis or longer depending on the water demand and available supply in the canal.

8.4. Previous Optimization Models for Canal Operations

A few attempts are reported in the literature to optimize canal gate controls with a focus on meeting irrigation demands. Soman and Hill (1989) developed a linear programming model to minimize the differences between user demands and the gate releases with user demands as the input for the model. Reddy et al. (1992) formulated an optimization model for canal gate control in the presence of arbitrary external disturbances based on a linearized finite difference model of open-channel flow. Lin and Manz (1992) developed a canal control methodology based on a non-linear programming model combined with a dynamic simulation model to assist operators in making optimum operational decisions for gate controls. Pongput and Merkley (1997) and Malaterre and Baume (1998) documented the different modeling and optimization approaches for irrigation canal controls and a comparison between the different approaches. Gómez et al. (2002) presented a digital control scheme for water level regulation in irrigation canals. The scheme considered a canal prototype with a series of pools connected with active gates. Wahlin and Batista (2003) explored the application of a feedforward control algorithm for canal gate control. Wahlin (2004) assessed the performance of a model predictive control algorithm for canal gate control for an example canal system. Previous approaches for canal control optimization consider only gate control and do not consider pump operations. The methodology presented herein considers both gate and pump controls for water supply canal systems (WSCS).

8.5. Mathematical Formulation of Optimization Model

The optimization model for WSCS is formulated as a steady state model that is solved at consecutive time periods for gate and pump operation of the system. The steady state optimization model is solved separately for each consecutive time-period of operation with the inputs updated each time-period.

8.5.1. Objective function

The purpose of the optimization model is to determine optimal pump and gate operations with the objective of satisfying requested discharges by minimizing the difference between requested and satisfied discharges at demand locations throughout the system. Emergency considerations include possible power shortages and/or available water shortages and other operational constraints. The objective function (Z) is to minimize the difference in requested flows ($Q_{req\ j}$) and supplied flows ($Q_{sup\ j}$) each time-period.

$$\text{Minimize } Z = \sum_{j=1}^{j=J}\left[\left(Q_{req\,j} - Q_{sup\,j}\right)^{n}\theta_j\right] \tag{8-1}$$

Where, $Q_{req\,j}$ is the requested flow at cross-section j, $Q_{sup\,j}$ is the supplied outflow at canal cross-section j, n is the exponent used for objective function where $n \geq 1$, and θ_j is a penalty multiplier for the requested canal discharge at cross-section j not being supplied.

Constraints of the model include continuity, conservation of energy, gate controls, pumping, electrical power availability, and bound constraints on water surface elevations and supplied discharges for a specified time period.

8.5.2. Continuity

The continuity equation for each reach r defined by channel cross-sections j+1 and j (see Figure 8-3) is given as

$$Q_{j+1} = Q_j + q_s L_r + Q_{sup,j} \tag{8-2}$$

Where, Q_j is the discharge at cross-section j and q_s is the evaporation rate per unit length of canal. The continuity equation for a pumping station between cross-sections j and j+1 is

$$Q_{j+1} = \sum_1^P Q_p = Q_j + Q_{sup,j} \tag{8-3}$$

Where P is the total number of pumps in the pumping station and Q_p is the discharge for each pump p.

8.5.3. Conservation of Energy

The conservation of energy for a canal reach is written in terms of water surface elevations (WSE_j) and velocity heads $\left(\frac{a_j (V_j)^2}{2g}\right)$ at upstream (j+1) and downstream (j) cross-sections of a reach, and the energy head loss for the reach is given as follows

$$WSE_{j+1} + \frac{a_{j+1}(V_{j+1})^2}{2g} = WSE_j + \frac{a_j (V_j)^2}{2g} + h_{e\,(r)} \tag{8-4}$$

Where WSE_{j+1} and WSE_j are water surface elevations, V_{j+1} and V_j are velocities and a_{j+1} and a_j are energy correction factors at upstream (j+1) and downstream (j) cross-sections for reach r; g is the gravitational acceleration; $h_{e\,(r)}$ is the energy head loss for reach r. The energy head loss ($h_{e\,(r)}$) is given as follows

$$h_{e\,(r)} = L_r\bar{S}_{f(r)} + C_r\left|\frac{a_{j+1}(V_{j+1})^2}{2g} - \frac{a_j(V_j)^2}{2g}\right| \tag{8-5}$$

Where L_r is the length of reach r, $\bar{S}_{f(r)}$ is the average friction slope for reach r and C_r is the expansion or contraction loss coefficient for reach r. The average friction slope is defined in terms of the conveyance (K_j) as $\bar{S}_{f(r)} = \left(\frac{Q_j+Q_{j+1}}{K_j+K_{j+1}}\right)^2$ where $K_j = \frac{1.486}{n}A_j\left(R_j\right)^{\frac{2}{3}}$. A_j is the cross-sectional area of flow and R_j is the hydraulic radius for cross-section j.

The energy equation for a reach with a pumping station is

$$WSE_{j+1} + \frac{a_{j+1}(V_{j+1})^2}{2g} + h_{p(r)} = WSE_j + \frac{a_j(V_j)^2}{2g} \tag{8-6}$$

Where, $h_{p\,(r)}$ is the total dynamic head of the pumps defined using the pump head-discharge relationship.

8.5.4. Pump Constraints

A simplified pump head-discharge relationship is

$$h_{p\,(r)} = c_{p,r}(Q_{p,r})^2 + a_{p,r} \tag{8-7}$$

Where $c_{p,r}$ and $a_{p,r}$ are coefficients for pump p in reach r.

Power consumed (Pow_p) by a pump p in reach r is given as follows

$$Pow_{p,r} = \frac{X_{p,r}\,Q_{p,r}h_{p\,(r)}}{3956\,\eta_p} \tag{8-8}$$

Where $Pow_{p,r}$ is the power consumed by pump p (in horsepower); $X_{p,r}$ is a variable defining the speed ratio (speed of pump/maximum speed) for a pump p in reach r and η_p is the efficiency of pump p in reach r.

8.5.5. Gate Control Constraints

The flow through gate control structures such as sluice gate is expressed as

$$Q_{gate,r} = C_d \, W_r \, B_r \sqrt{2g(WSE_{j+2} + \frac{a_{j+1}(V_{j+1})^2}{2g} - WSE_j - \frac{a_j(V_j)^2}{2g})} \quad (8\text{-}9)$$

Where $Q_{gate,r}$ is the flow through the channel cross-section at a sluice gate in reach r; C_d is the coefficient of discharge; B_r is the decision variable for height of gate opening and W_r is the width of the gate opening. Equation 8.9 can be applied for both free flow and fully submerged flows with the coefficient of discharge (C_d) ranging from 0.5 to 0.7 for free flow conditions and 0.8 for fully submerged conditions.

8.5.6. Power Availability Constraint

The power availability constraint is defined in terms of an upper bound for power consumption as

$$\sum_1^P \frac{X_{p,r} Q_{p,r} h_{p\,(r)}}{3956 \, \eta_p} \leq Pow_{availble,r} \quad (8\text{-}10)$$

Where $Pow_{availble,r}$ is the total power available at pumping station in reach r.

8.5.7. Bound Constraints

Upper and lower bounds for water surface elevations (WSE_j) are required at each cross-section j in the canal system are

$$\underline{WSE_j} \leq WSE_j \leq \overline{WSE_j} \quad (8\text{-}11)$$

Where, $\underline{WSE_j}$ and $\overline{WSE_j}$ are lower and upper bounds for water surface elevations at cross-sections j, respectively.

Upper and lower bounds for the supplied discharges ($Q_{sup\,j}$) at each cross-section j are

$$\underline{Q}_{sup\,j} \leq Q_{sup\,j} \leq Q_{req\,j} \quad (8\text{-}12)$$

Where, $\underline{Q}_{sup\,j}$ is the lower bound for supplied outflows at channel cross-section j.

8.6. NLP Solution Methodology for Example Application

For a simple canal system, the nonlinear programming model (equations 1-12) can be solved using MINOS (Murtagh and Saunders, 1982) in the general algebraic modeling system (GAMS) (Brooke et al., 2005) for each hour of WSCS operations. $X_{p,r}$ and B_r are decision variables and $Q_{sup\,j}$, WSE_j, $Q_{p,r}$, $Q_{gate,r}$, $Q_{p,r}$, $h_{p\,(r)}$ and V_j are state variables. The example canal system has nine cross-sections (Figure 8-3) with a gated inline structure at cross-section 4 and a pumping station at cross-section 7.

Table 8-1 gives the details of the first example canal system including bed elevations (Z_j), cross-section type, reach number, upstream and downstream cross-section for reaches and length of the reach (distance between cross-sections). The example canal has a uniform bed slope of 0.00015 with a rise of 5 ft between cross-sections 7 and 8. A low head pumping system (consisting of two variable speed pumps) is between cross-sections 7 and 8 to facilitate uninterrupted flow. Cross-sections 2 and 6 are the demand locations with requested outflows as a function of time (see Figure 8-4).

Table 8-1 Details of Example Canal System 1

Channel C/S (J)	1	2	3	4	5	6	7	8	9
Cross-section type	D/S	C/S	C/S	Gate	C/S	C/S	Pump	C/S	U/S
Elev. (ft)	0	1.98	3.96	3.96	3.97	5.95	5.98	0.97	3.95
Bottom width (ft)	10	10	10	10	10	10	10	12	12
Reach	1	2	3	4	5	6	7	8	
Downstream C/S	1	2	3	4	5	6	7	8	
Upstream C/S	2	3	4	5	6	7	8	9	
Reach length (ft)	13200	13200	50	50	13200	50	50	13200	

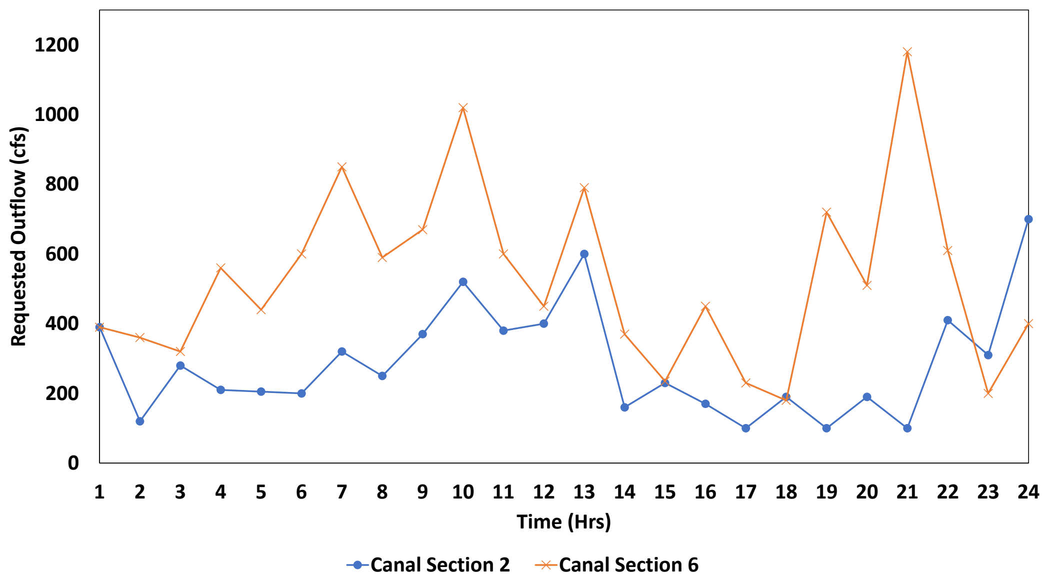


Figure 8-4 Hourly Requested Outflows at Canal Sections 2 and 6 for Example Canal System 1

A scenario of limited water availability (Figure 8-5) was considered for this application. The model was applied for a water shortage scenario for each hour of the 24-hour operations of the WSCS. Figure 8-5 shows the results of the NLP model application for the example canal system including the total supplied outflows ($Q_{\mathrm{sup}\,j}$), total requested outflows ($Q_{\mathrm{req}\,j}$), flow and the objective function (Z). The trends of the supplied and requested outflows show that the optimized canal controls facilitate maximum possible supply to satisfy the requested outflows while keeping the total supply within the upper bound of limited available flow. Figure 8-6 shows optimized gate openings for the 24-hour operations of the example canal system. The gate openings are smaller during times of lower flow availability in order to maintain the required water surface elevations on the upstream. Figure 8-7 and Figure 8-8 show the pump discharges and the power utilized. The pumps are operated for minimum power consumption while meeting the downstream flow requirements.

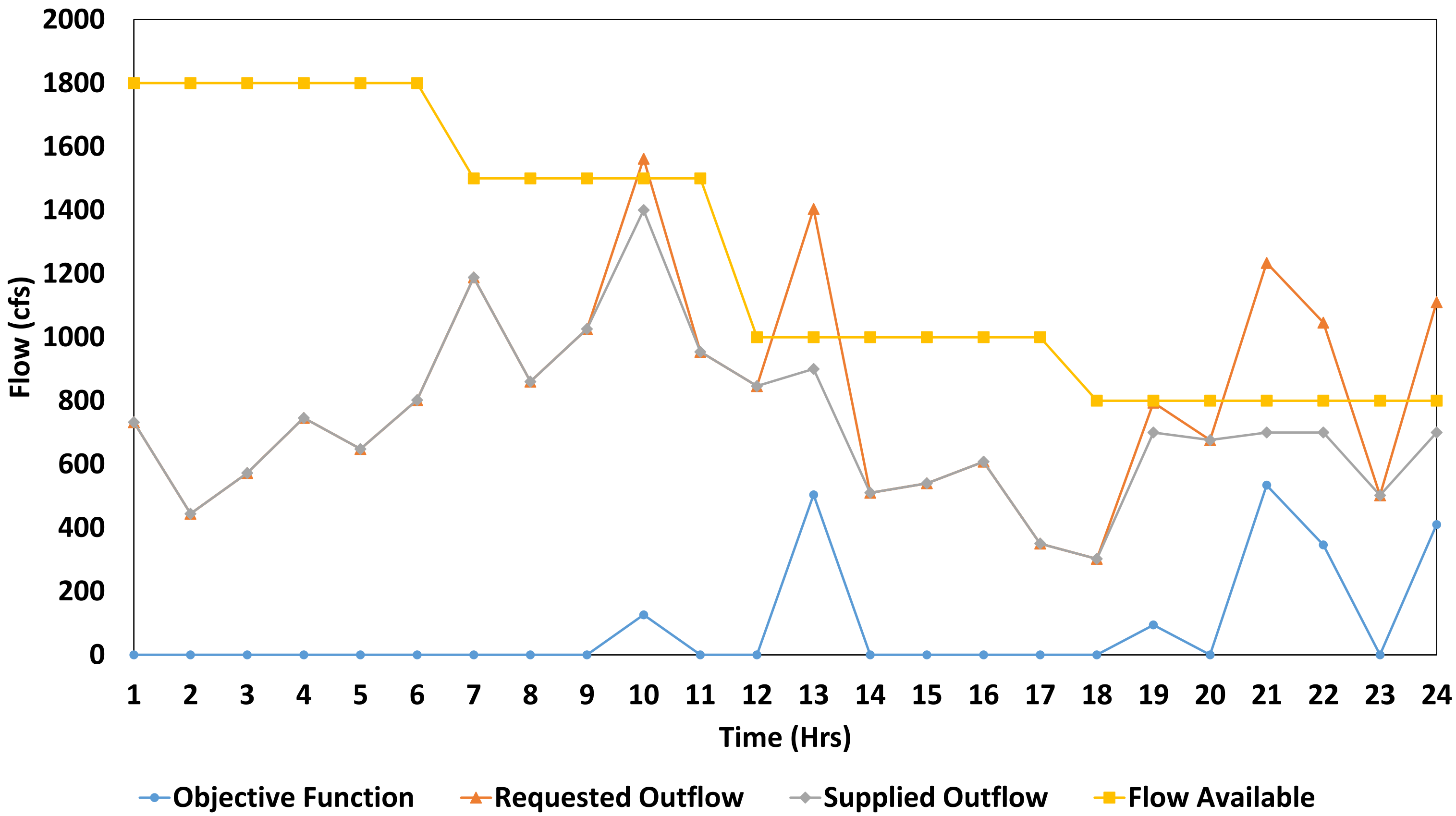


Figure 8-5 Model Application Results for Example Canal System 1

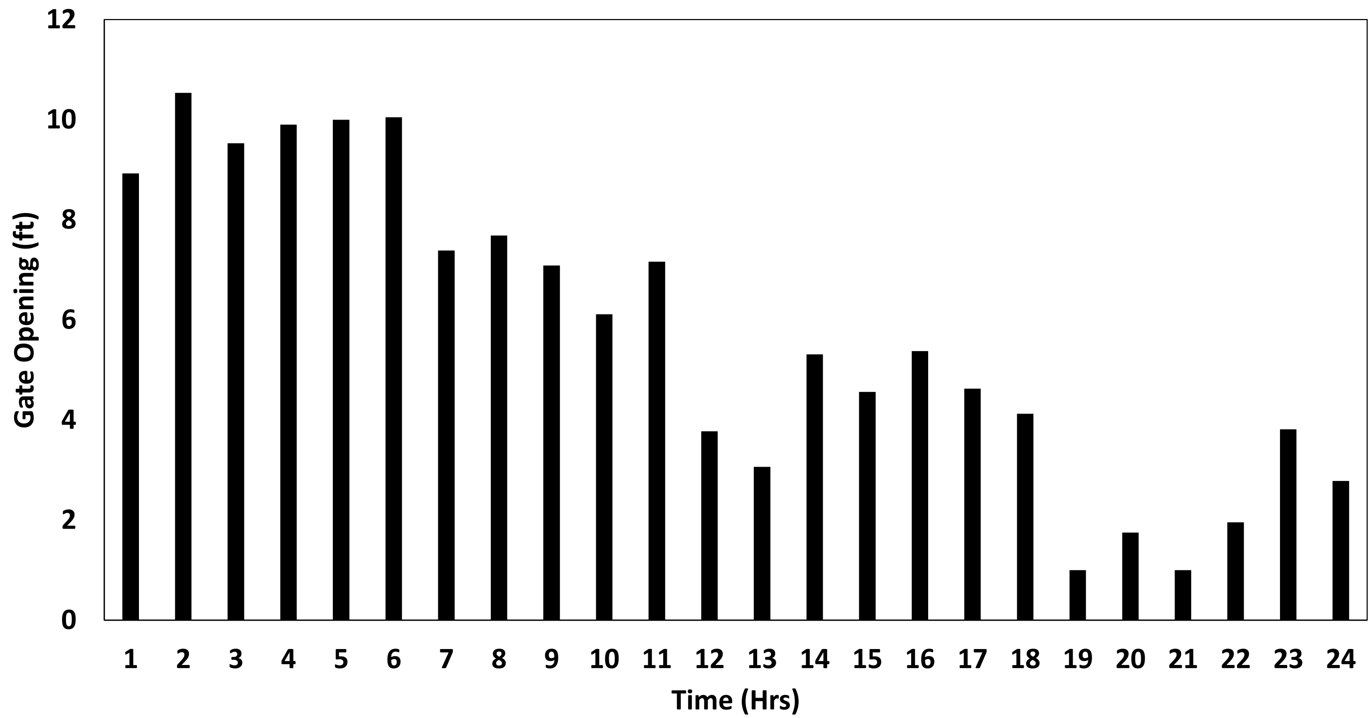


Figure 8-6 Hourly Optimal Gate Openings for Example Canal System 1

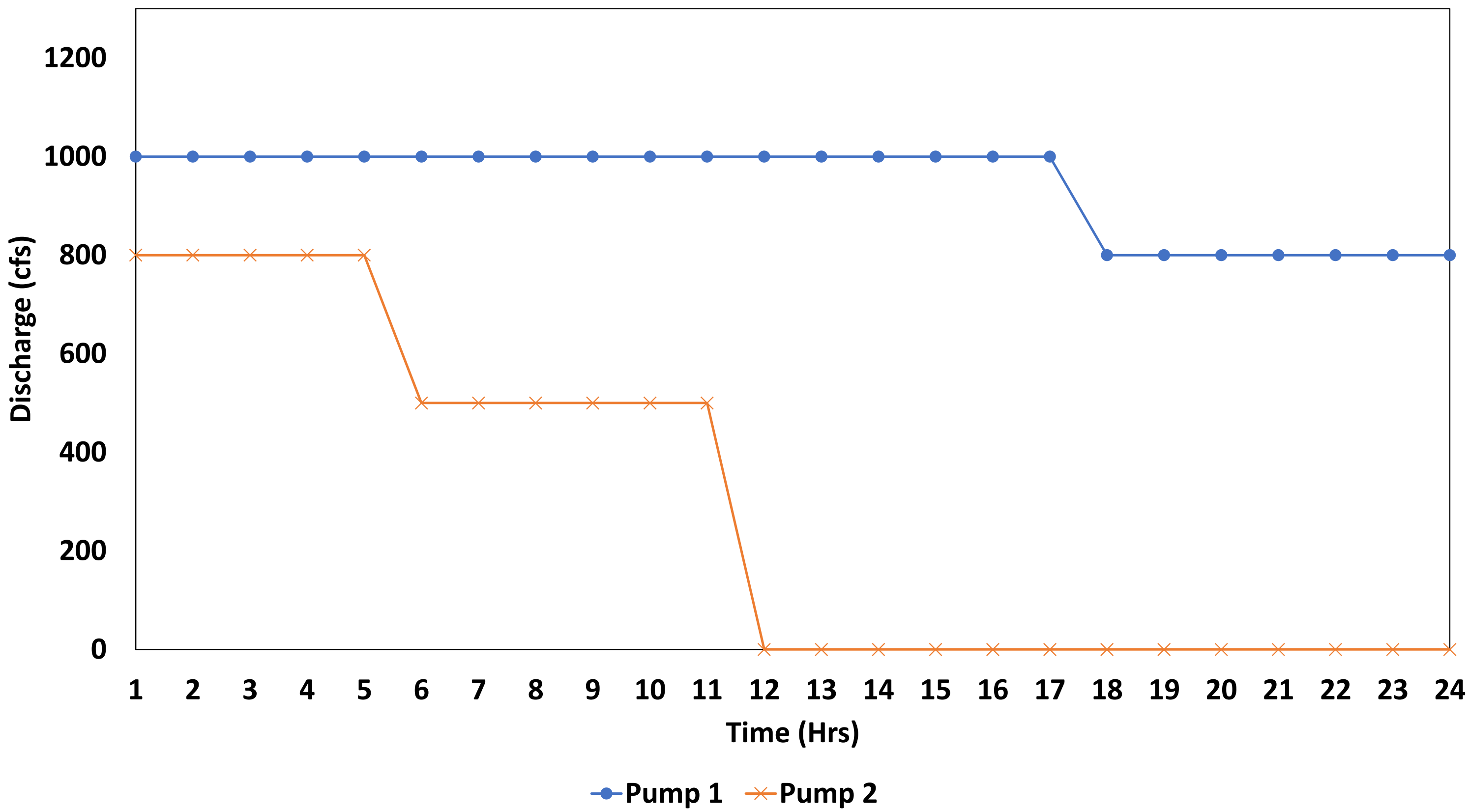


Figure 8-7 Optimal Pump Discharges for Pumping Station at Cross-section 7 for Example Canal System 1

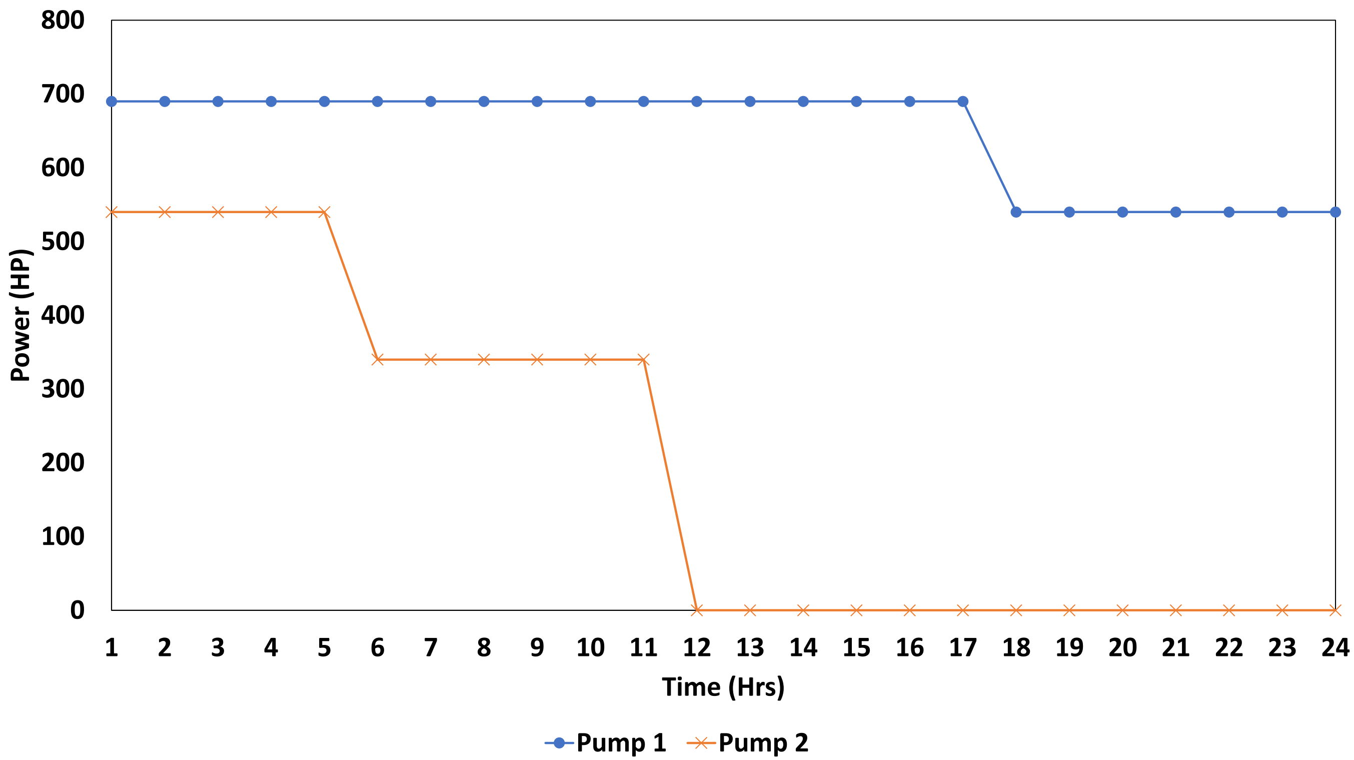


Figure 8-8 Power Consumption for Two Pumps at Cross-section 7 for Example Canal System 1

8.7. Optimization-Simulation Approach

The optimal control problem is solved using an optimization – simulation approach for determining the WSCS optimal gate and pump controls for a complex branching canal system. This approach solves the optimal control model using the genetic algorithm in MATLAB (Chipperfield et al., 1994) interfaced with the open channel flow hydraulic simulator (HEC-RAS), which models both steady and unsteady flow. The HEC-RAS model is used to solve the hydraulic constraints for steady, gradually varying open channel flow for a certain time-period. The model is solved repeatedly for future time-periods for real-time operations of the WSCS.

8.7.1. HEC-RAS Simulation Model

The U.S. Army Corps of Engineers Hydraulic Engineering Center's (HEC) River Analysis System (HEC-RAS, Brunner, 2010; Goodell and Brunner, 2014) is a computer program for modeling the open channel hydraulics of water. It is accepted as a reliable hydraulic model by federal agencies including the Federal Emergency Management Agency (FEMA), the National Weather Service (NWS), the National Resource Conservation Service (NRCS) and the U.S. Army Corps of Engineers (Goodell and Brunner, 2014). HEC-RAS version 5.0 can be used to model one and/or two-dimensional, steady or unsteady flow. One dimensional steady - state flow modeling was used for the purposes because the canals are prismatic, and controls are considered on a time-frame that does not require unsteady flow modeling for each time period.

The basic computational procedure for HEC-RAS one dimensional steady - state modeling is solving the one-dimensional energy equation. Energy losses for friction are based upon Manning's equation. Boundary conditions define the starting water surfaces throughout the canal. HEC-RAS uses internal boundary conditions for connections to the junctions for gate control structures and pumps. Typically, boundary conditions are user supplied flows on the upstream and downstream of the canal reach (Brunner, 2010). Gates and pumps in the canal as well as outflows from the canal are defined by internal boundary conditions, while the upstream and downstream starting water surface elevations are defined.

8.7.2. <u>Reduced Optimization Model</u>

A genetic algorithm solves an unconstrained optimization problem which requires a reduced mathematical model including bound constraints (equations 8.9 – 8.12) for the optimization formulation. The following reduced objective function is solved using the genetic algorithm.

$$Z_{\mathrm{reduced}} = Wp_1 \sum_{j=1}^{j=J}\left[\left(Q_{\mathrm{req}\,j} - Q_{\mathrm{sup}\,j}\right)\theta_j\right] + Wp_2 \sum_{j=1}^{j=J}\left[w_1 \max\left(0, \underline{WSE_j} - WSE_j\right) + w_2 \max\left(0, WSE_j - \overline{WSE_j}\right)\right] + Wp_3 \sum_{j=1}^{j=J}\left[wq_1 \max\left(0, \underline{Q}_{\mathrm{sup}\,j} - Q_{\mathrm{sup}\,j}\right) + wq_2 \max\left(0, Q_{\mathrm{sup}\,j} - Q_{\mathrm{req}\,j}\right)\right] + Wp_4 \max\left[0,\ \sum_{1}^{P} \frac{x_{p,r}\, Q_{p,r}\, h_{p\,(r)}}{3956\, \eta_p} - Pow_{\mathrm{availble},r}\right] \tag{8-13}$$

Where, Z_{reduced} is the reduced objective function; Wp_1, Wp_2, Wp_3 and $Wp_4$are penalty weights for the objective function, water surface elevation bounds, supplied outflow bounds and upper bound for pump power consumption, respectively; w_1 and w_2 are additional penalty weights for lower and upper bounds for water surface elevations respectively; wq_1 and wq_2 are additional penalty weights for lower and upper bounds for supplied outflow bounds respectively.

The penalty weights (Wp_1 - Wp_4) and the exponent (n) for the objective function are determined through a sensitivity analysis for each application of the optimization model. The value of weights depends upon the relative importance of the penalty term and the numerical value of the penalty term. The reduced optimization model is tested for different values of penalty weights as well as different combinations of the penalty weights to obtain values of penalty weights that provide the minimum value of the reduced objective function and the highest penalty satisfaction. The values of penalty functions $Wp_1 = 10$, $Wp_2 = 10^4$, $Wp_3 = 10^8$ and $Wp_4 = 10^3$ are used for the example application of the optimization-simulation model. The values for additional penalty weights used are $w_1 = 1$, $w_2 = 1$, $wq_1 = 1$ and $wq_2 = 1$. The value of n = 1 is used for the example application.

The hydraulic constraints (equations 8.2 – 8.8) are solved implicitly by HEC-RAS for each iteration of the genetic algorithm that solves the reduced optimization model (equation 8.13) through the MATLAB-VBA-HEC-RAS interface/framework (see Figure 8-9). The interfacing of the genetic algorithm in MATLAB and HEC-RAS simulator was achieved through Excel VBA as shown in Figure 8-9. Detailed information on VBA coding for HEC-RAS is given by Goodell and Brunner (2014).

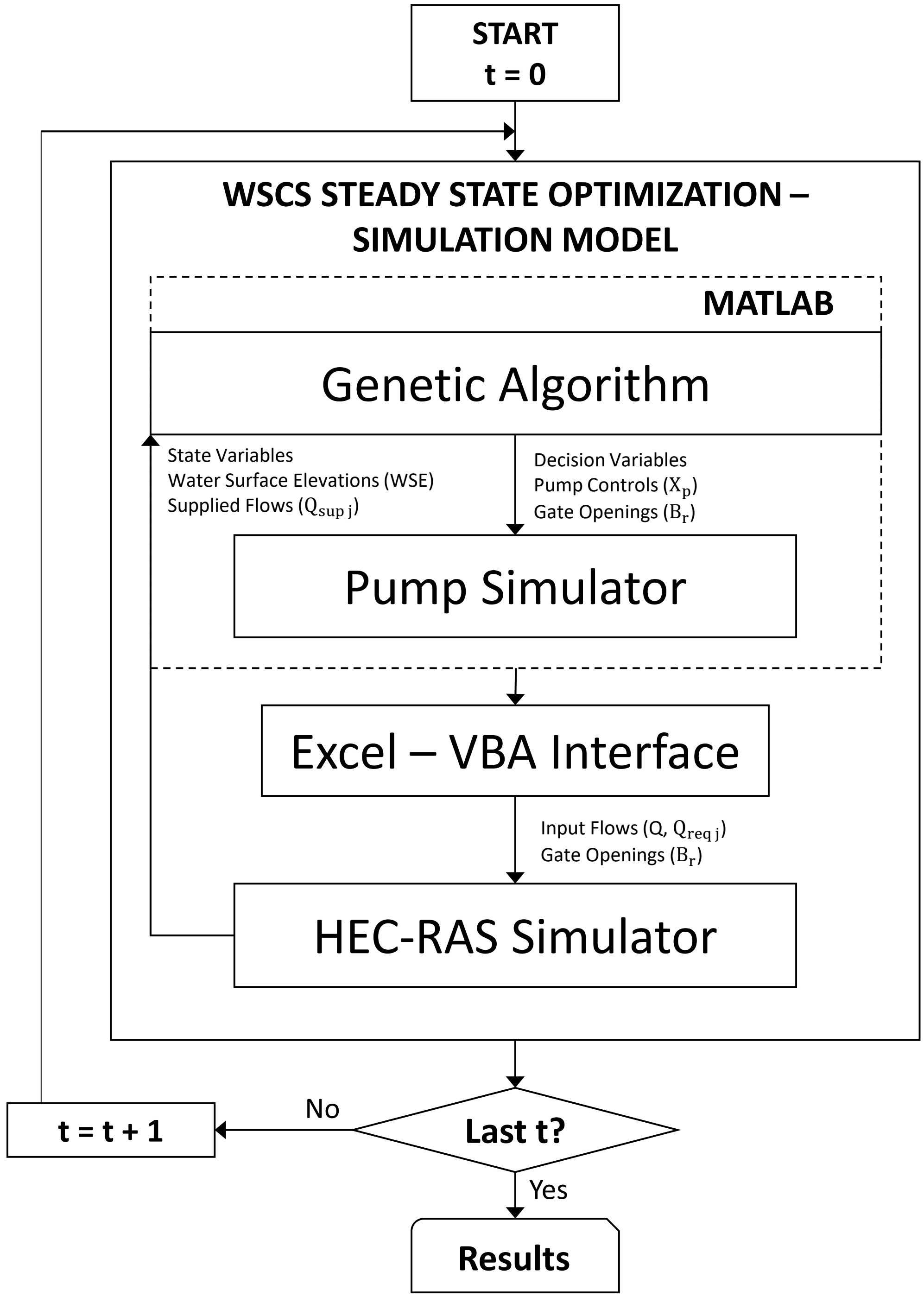


Figure 8-9 Optimization – Simulation Framework

8.8. Example Applications of Optimization-Simulation Approach Considering Limited Power Availability

A second example canal system (shown in Figure 8-10) was used to demonstrate the application of the reduced optimization model given in equation 8.13 using a MATLAB–VBA–HEC-RAS interface developed as a part of this study. The example canal system is a diverging type of canal system which serves four cities (see Figure 8-10). The details of the canal system including the cross-sections, bed elevations, type of cross-sections and downstream reach lengths are given in Table 8-2. The example canal system 2 consists of two offtake canals, which provide flow to the storage facilities for the four cities. Each offtake canal is provided with a pumping station to pump water from the main canal, which is at a lower elevation than the offtake canal. The main canal splits into two reaches at cross-section 2 (a mile downstream from the upstream cross-section 3). Each of the two pumping stations have two variable speed pumps operating in parallel. Figure 8-11 shows the head-discharge curves for the pumps.

Table 8-2 Details of Example Canal System 2

Channel	Cross-section	Type of cross-section	Elev. (ft)	Bottom width (ft)	Downstream reach length (ft)
Main canal	3	Upstream CS	16	8	10560
Main canal	2	Outflow CS	8	8	5280
Main canal	1.943	Gate 2	7.93	8	4980
Main canal	1.61	Pumping station 2	4.89	8	3221
Main canal	1	Downstream CS	0	8	0
Main canal split	2	Split upstream CS	8	8	5280

Main canal split	1.2	Gate 1	1.78	8	1056
Main canal split	1.11	Pumping station 1	0.89	8	581
Main canal split	1	Split downstream CS	0	8	0
Offset canal 1	2	Upstream CS	30	8	1000
Offset canal 1	1	Downstream CS	28	8	0
Offset canal 2	2	Upstream CS	29.89	8	5280
Offset canal 2	1	Downstream CS	21.89	8	0

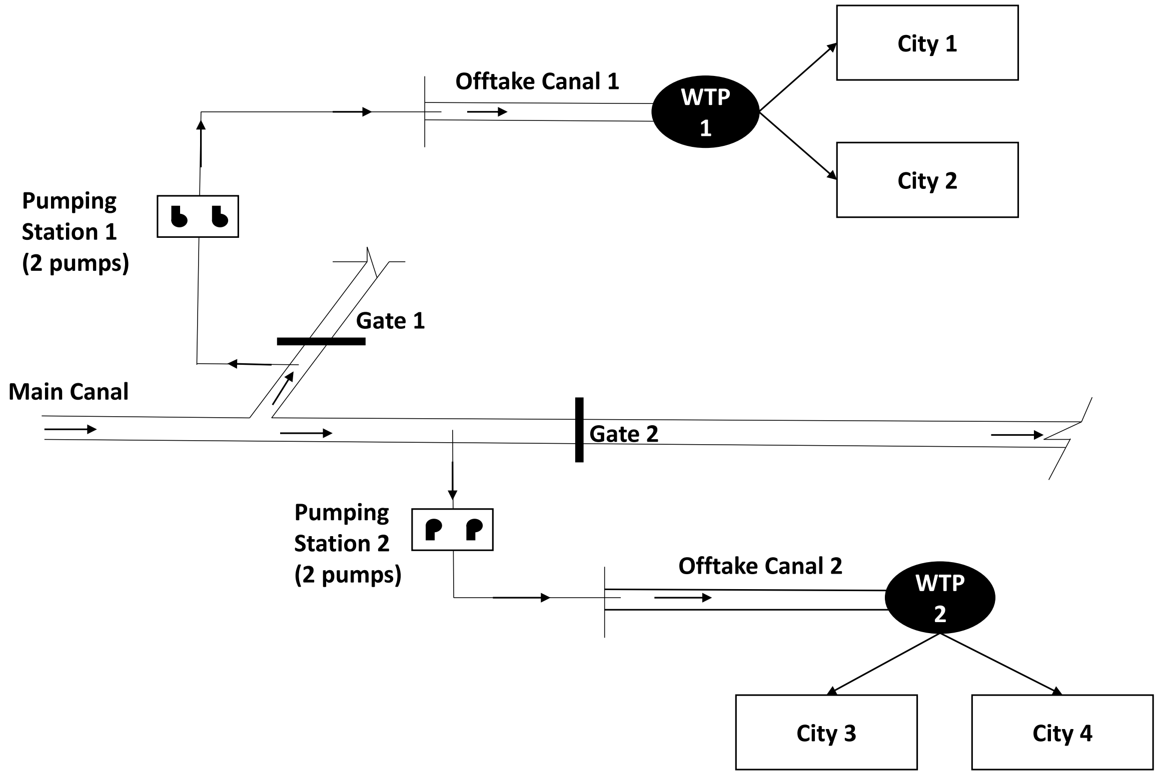


Figure 8-10 Example Canal System 2

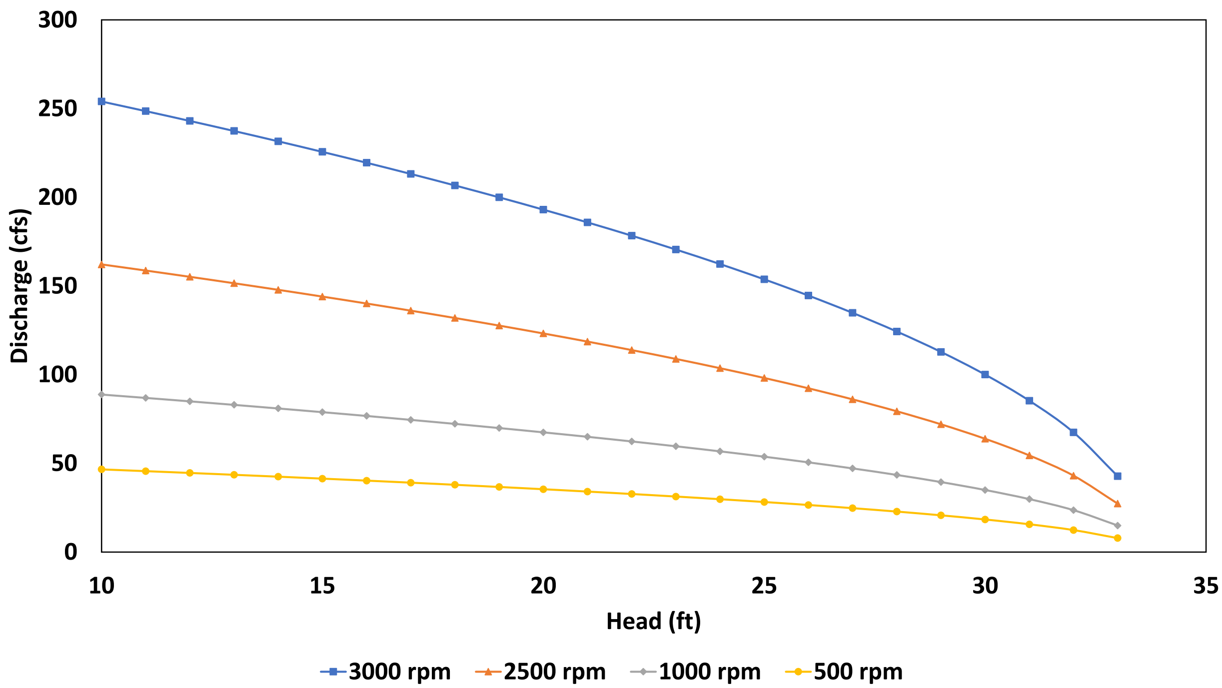


Figure 8-11 Pump Curves for Variable Frequency Pumps for Example Canal System 2

The optimization-simulation methodology was applied for a scenario of limited power availability. Figure 8-12 illustrates the power availability and consumption for the optimized operations of example canal system 2. The pumps were operated such that the power consumed is less than the power available ($\mathrm{Pow}_{\mathrm{availble}}$). Figure 8-13 (a) and (b) show the requested and supplied flows for the two offtakes. The requested outflows are satisfied for all the 24 hours of model application except for the times of limited power availability. Figure 8-14 illustrates the optimized gate openings so that ~~a~~ sufficient flows and water surface elevations are maintained at the intakes of the pumping stations. Figure 8-15 shows the optimized pump speeds.

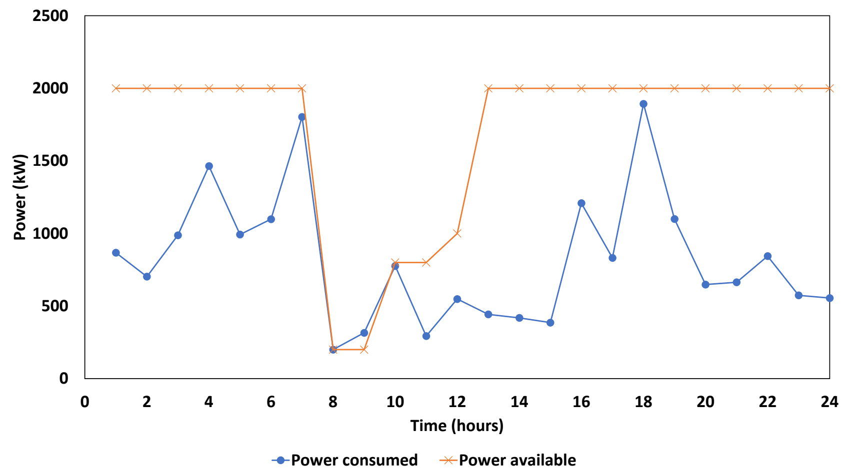
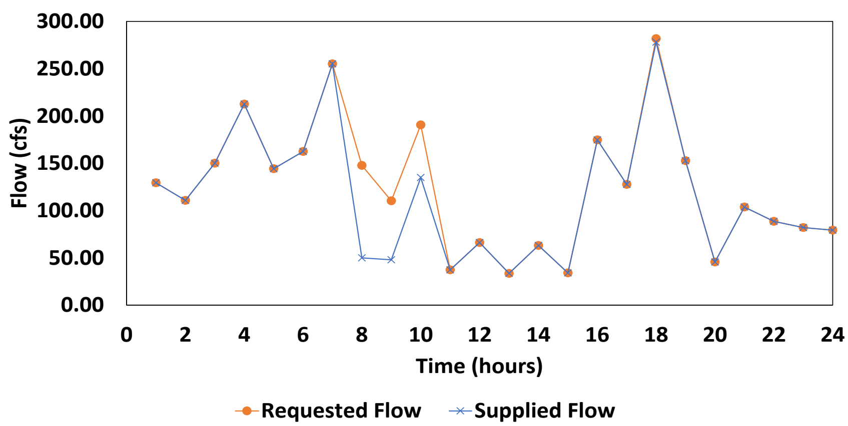
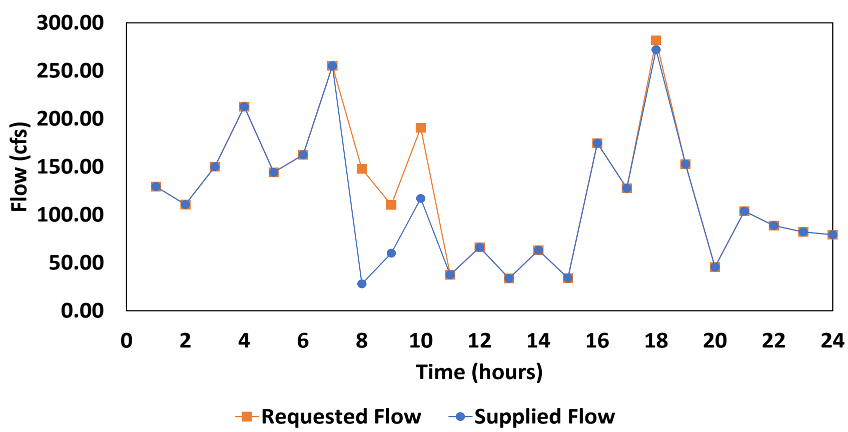


Figure 8-12 Power Availability and Consumption for Example Canal System 2



a. Offtake canal 1



b. Offtake canal 2

Figure 8-13 Requested and Supplied Flows at Offtake Canals for Example Canal System 2

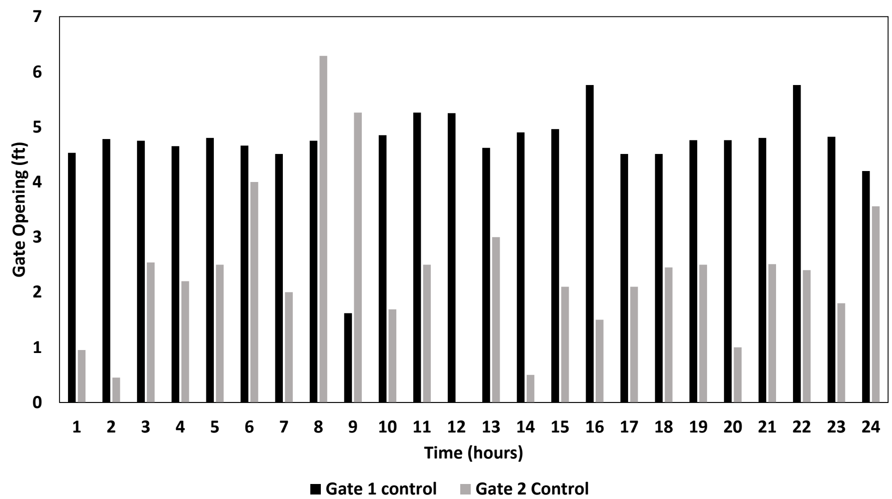
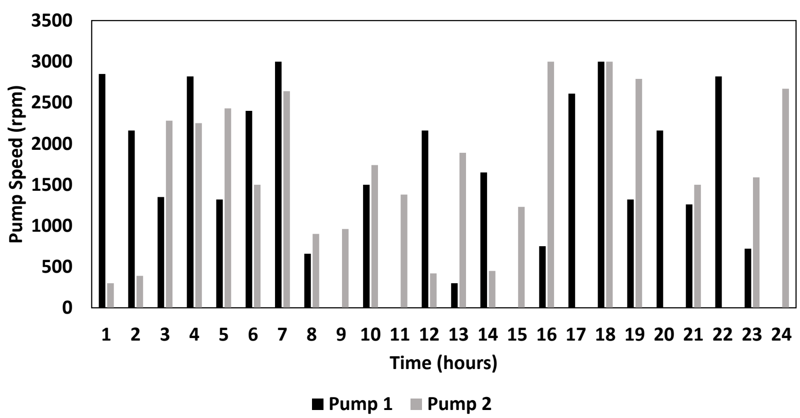
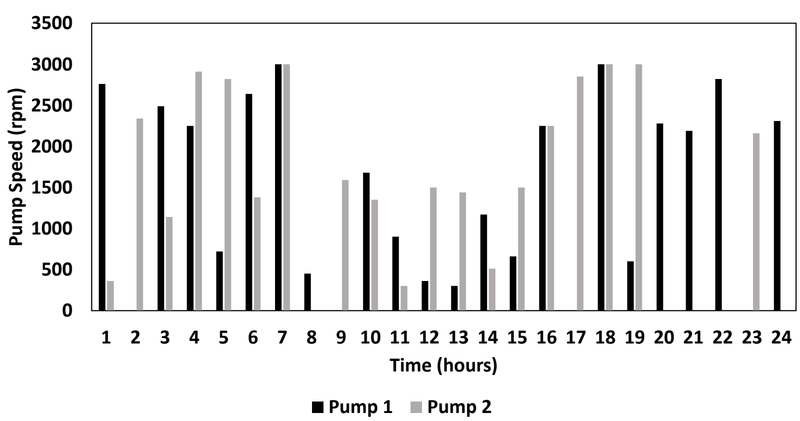


Figure 8-14 Optimized Gate Controls for Example Canal System 2



a. Pumping Station 1



b. Pumping Station 2

Figure 8-15 Optimized Pump Speeds for Pumping Stations for Example Canal System 2

8.9. Conclusions

Two new methodologies, NLP optimization model and an optimal control approach using the optimization-simulation model, for optimal operations of water supply canal gates and pumps under limited power and/or water availability are developed. The methodologies are novel for two reasons: (1) introduction of pump and gate controls in the canal control optimization framework and (2) consideration of limited power and water availability.

The non-linear programming (NLP) model is a simplified method for the canal gate and pump control optimization without use of a simulation model. Application of the NLP methodology is limited to simple canal systems with a limited number of reaches and diversions. The second methodology is an optimization-simulation approach using the genetic algorithm in MATLAB interfaced with HEC-RAS. The hydraulic modeling capabilities provided by HEC-RAS and mathematical simplicity of the reduced optimization model solved using a heuristic approach (genetic algorithm in MATLAB) provide for a platform to model and optimize the real-time pump and gate controls of a complex branching WSCS.

The results of application of NLP solution methodology for the simple canal system show satisfaction of the requested demands except the times of limited water availability. During time-periods of limited water availability, the model allocates the water to meet the requested demands to the maximum possible extent while maintaining the required minimum water surface elevations (WSE_j) in the channel. The optimization – simulation approach was applied to a more complex WSCS including four cities with a main canal, offtake canals, and pumping stations, with limited power availability at the pumping stations. The model supplied the requested to the maximum possible extent the limited available power at the pumping stations. The applicability of the WSCS pump and gate control methodology as a decision-making tool for WSCS operations during emergency conditions is evidenced from the example applications.

Emergency conditions such as droughts can lead to chronic water shortages, reducing the capacity of a regional WSCS to supply water in required quantities. WSCS, WDS and EPS are highly interdependent and could fail in a cascading manner in case of emergency operations during limited water and/or power availability. Future research will include interfacing of the WSCS operations methodology with a regional WDS optimization-simulation model and a EPS optimization-simulation model to assess the effects of water shortage on electrical power generation and water supply.

9. ADAPTIVE MANAGEMENT FOR CONTINGENCY SCENARIOS

9.1. Adaptive Management for Governance of Emerging Technologies

The technologies that are already conceptualized or not yet conceptualized which are foreseen to be materialized in a reasonably near future may be termed as 'emerging technologies'. An uncertainty exists with regards to the benefits and undesirable side effects of such foreseen or unforeseen technologies, which is evident from the definition formulated here, that depicts a lack of complete understanding of such technologies. Wiek et al. (2007) highlight this uncertainty while stating the importance of governance of emerging technologies to avoid the various unwanted and disastrous side effects. Sustainable governance of emerging technologies addresses this issue from an integrated societal perspective that proposes collaboration among agents from science, business, government, and the public during the process of technological innovation and diffusion (Wiek et al., 2007). The onus of policy making with regards to technological development and innovation has gone through a change in its focus from enhancement of economic benefits from such technologies to tackling societal challenges and achieving an overall environmental sustainability.

Allenby (2014) suggests the use of adaptive management as a principle for governance of the emerging technologies. One of the classic ecological definitions of adaptive management is 'ways for active adaptation and learning in dealing with uncertainty in the management of complex regional ecosystems' (Gunderson et al., 1995; Allenby, 2014, p. 344). This definition highlights the iterative adaptation and learning process involved in any problem-solving technique for an uncertain problem, while also addressing the 'complexity of regional ecosystems'. Though this definition addresses the ‘ecological’ part of the problem, it fails to address the other undesirable effects that an emerging technology may have on the society, which is a multi-faceted and complex system. Any complex system is prone to cascading beneficial or non-beneficial effects from an interference. This increases the uncertainty in the effects of an emerging technology on the society at large.

A more generic definition of adaptive management, as given by Williams et al. (2007, p. 2) states, ‘Adaptive management is a systematic approach for improving resource management by learning from management outcomes’. This simple definition gives a better and more holistic understanding of the term. A ‘systematic approach’ provides for a well-defined plan, while ‘management by learning from management outcomes’ shows the iterative nature of this technique. To simplify further, adaptive management is a technique of managing a complex problem doomed with uncertainty by a continuous learning or iterative approach.

9.2. Performance Evaluation of the Optimization-Simulation Methodology through Quantitative Metrics

Water and energy distribution systems are critical infrastructures for any civilized area. These two systems, collectively, are known informally as the water-energy nexus. Extreme conditions like droughts, hurricanes, tornadoes, earthquakes are major disruptions in normal operations of the water – energy nexus. In addition to these several short-term contingencies could disrupt the operations including but not limited to power plant failure, failure of pumps, pipes or other components of the water distribution system (WDS), routine maintenance of both the water and power systems etc. A mathematical optimization/simulation model was developed for the purpose of maximizing the demand satisfaction as well as prioritizing the various demands within the scope of a WDS considering the interdependencies between the water and energy systems was presented as a solution for the problem of operating the WDS during such contingencies and emergencies. The methodology considers a continuous learning approach wherein its application could be easily modified for any new data available at a future time.

Evaluation of the technological fix proposed in the form of the combined energy – water optimization/simulation model, could be performed by applying the model to a variety of predicted futuristic scenarios such those proposed by Intergovernmental Panel on Climate Change (IPCC) (IPCC, 2007), extreme drought conditions predicted for US southwest by NASA (Cook et al. 2015), predicted futuristic power demands and cooling water demands for power plants (Koch and Vögele, 2009) etc. Larson et al. (2009) give the specific assessment of the water demand and scarcity predictions for Phoenix Metropolitan area, which could be used as a representative of the American Southwest drought scenario. The optimization models developed should be tested for simulations of real or realistic-hypothetical water distribution systems to evaluate their functioning. Along with the short-term operation changes proposed by use of the mathematical models, a few system improvements would also be required as a long term investment for implementation of the methodology proposed in this project. These long-term investments involve important functional and design changes such as inclusion of flow control valves, storage components etc in the WDS, which are also important for quality assurance and robustness of the overall system. The mathematical models were developed assuming minimum improvements are achieved, but such changes would achieve considerable improvement in the performance of the WDS.

Specific quantitative metrics used to determine the overall performance of the proposed methodology include demand satisfaction (ratio of demands satisfied to the required demand), pressure constraint violations, power constraint violations, consumer satisfaction, etc. The quantitative metrics used here to ascertain system performance are instrumental in ensuring an incremental and reversible alteration to the WDS. While developing this methodology, a consideration is that the solution should be a part of the system rather than an externality. The control methodology developed here would make use of the existing system while expecting emergence of the system as a whole as well as its components. The methodology considers unexpected population growths leading to unpredicted increases in demands in the various parts of the system. It is also designed not to be dependent on the artificial boundaries and thus accepts any unforeseen expansion of the WDS within reasonable natural boundaries. Another important consideration for ensuring the performance of a WDS is the reliability of the system. Mays and Tung (2002, p. 185) give a detailed methodology for computation of the reliability of WDS operations. A load – resistance interference approach for reliability computation is used in this case to ensure a reliable system rather than increasing the redundancy of the hardware of the system.

9.3. Strategies and Scenarios for Water Distribution Systems Management Under Extreme Droughts and Limited Power Availability

Water and energy distribution systems have several interdependencies, which in current practice are not leveraged for optimization or for emergency operations. The mathematical models and the software packages currently under development would be instrumental in providing a multi-faceted solution for this problem. The model would optimize the operations of both the systems to maximize the demand satisfaction and minimize the operation costs under normal operations. The second facet of the methodology involves a mathematical algorithm that generates an operation schedule for the Water Distribution System (WDS) to deal with short term emergency situations like powerplant / power grid failure, short term power outages etc. The third part of the methodology deals with long term contingencies like droughts, long term power shortages, etc, which is dealt with by making long term operation changes within the distribution system. The second and third part of the WDS optimization/control methodology is brought into action only in case of a emergency situation, thus achieving a targeted intervention. Table 9-1 lists the various situations in which an intervention would be required in the operations and the hardware of WDS for various scenarios. Table 9-1 just lists a summary of a few scenarios. A more detailed description is proposed to be presented at a later phase of the project. The scenarios that would be discussed herein would be considered in various protocols based on their occurrence probability and risk values. Allenby (2014, p. 133) defines risk as ‘a probability of an event not necessarily but usually negative, occurring’. He goes further to associate risk with the losses caused by such an event in the following equation (Allenby 2014, p.133)

$$\text{Risk} = (\text{Probablity of an event}) \times (\text{losses associated})$$

Though this definition gives a correlation between the probability of the even occurring and the associated losses, it fails to address the extent of losses, the changes to the landscape and agencies caused by it as well as the efforts required to revive a system if such an event occurs. A more specific definition of risk would be used in determining the scenarios and the applicable protocols for this project. A risk analysis was performed to assess the risk of a occurrence of a certain scenario.

Water distribution systems vary in their operations and components across the globe. Though the various basic components and operating principles are universal, there are several design and topographic conditions which change from one system to another. Some water distribution systems are designed for an intermittent flow while majority of them are designed for a continuous supply. Also, the system might consist of several demand and pressure zones having different demand and head requirements. These among several others are the real-world boundaries which are considered while applying the real-time control methodology for any particular WDS. Another important real-world boundary for the water – energy nexus is that there is no central control or explicit control over the entire system, but a network of several control points, which are to be managed implicitly through a middleware. Apart from these there are several social and economic boundaries which would be required to be addressed through a multi-dimensional dialogue between the different stakeholders involved. In the case of a WDS emergency, dialogue with the stakeholders could be achieved through public communication systems such as television broadcasts, radio broadcasts etc. The major stakeholders involved in functioning of a WDS include the various public governing bodies such as Water agencies, local governments etc.; consumers including residents, businesses etc.; power companies and agencies; private operators etc.

Table 9-1 Targeted Intervention Using Control Optimization-Simulation Techniques

Contingency Protocol	Scenarios	Trigger Point	WDS operations / hardware modifications required (Remedial action required)
1. Short term limited power availability at not more than 10% of water facilities (STLPA 1)	Power availability is affected by a short term contingency such as a storm, component failure, vandalism and terrorist threats causing a disruption in the normal functioning etc.	Limited power availability predicted for not more than 10% of water pumps for a period of not more than 2 – 4 hours depending upon the WDS storage capacity.	1. No Hardware Modifications required. 2. Operation modifications include pump schedule modifications using the model developed for WDS operations under limited power availability considering pump operations only. 3. No effect on demands.
2. Short term limited power availability at more than 10% of water facilities (STLPA 2)	Power availability is affected by a short term major component failure leading to a reduced power availability at more than 10% of the water pumping facilities. This may be because of failure of the electric grid in a certain area of the city, terrorist attacks, extreme natural / manmade disasters affecting the WDS.	Limited power availability predicted for more than 6 - 10% of water pumps for a period of not more than 2 – 4 hours depending upon the WDS storage capacity.	1. No Hardware Modification required 2. Operation modifications include pump schedule modifications using the model developed for WDS operations under limited power availability considering demand satisfaction. 3. Demands of low priority zones within the WDS might have temporary effects.

3. Short term limited power availability at more than 50% of water facilities (STLPA 3)	Extreme scenario where the complete power grid for the area is affected leading to power failure to more than 50% of the pumps within the jurisdiction. Eg. Events like cascading grid failure, extreme storms leading to short term failures, extreme floods, etc.	Limited power available at more than 50% of water facilities for not more than 2 – 4 hours depending upon the WDS storage capacity.	1. Manual / automatic closure of valves to certain low priority zones required 2. Curtailed supply to power-plants and industrial zones for the contingency period. 3. Operation modifications include pump schedule modifications using the model developed for WDS operations under limited power availability considering pump operations and demand satisfaction. Reduced weights for demand. satisfaction constraints for low priority zones. 4. Demand satisfaction reduced for areas of lower priority until the system operations and storage is restored to normalcy.
4. Uncertain Water Demands for long term operations (UWD1)	Tourist locations, pilgrimage centers, agricultural areas, etc. have a great uncertainty in water demands. Climate change is also one of the major reasons leading to uncertain demands.	Uncertainty (> 15%) observed in the water demands of a water distribution system over a long-term run (more than 2 years consecutively)	1. Use of the stochastic MINLP model developed for uncertain water demands to obtain optimized pump schedules for daily operations of the pumps of the WDS. 2. No Design changes required.

5. Short term limited water availability because of component failure (STLWA1)	Failure of certain components of the water distribution system which feed the raw water to the water distribution system (such as canals, dams etc) to perform their valued function or of those components which are responsible for water supply to certain regions of the distribution system (such as pipes, pumps etc).	Temporary (not more than 6 hours) failure of an important component of the WDS that has a valued function of water supply to a part of the system.	1. Use optimization-simulation model given in Chapter 4 model for obtaining an optimized demand satisfaction pattern to meet the maximum possible water demands as per the priority of the various zones within the distribution system. 2. No design changes required except inclusion of flow control valves for the various water mains supplying water to the various zones within a city as well as the various supply areas of the overall regional water distribution system.

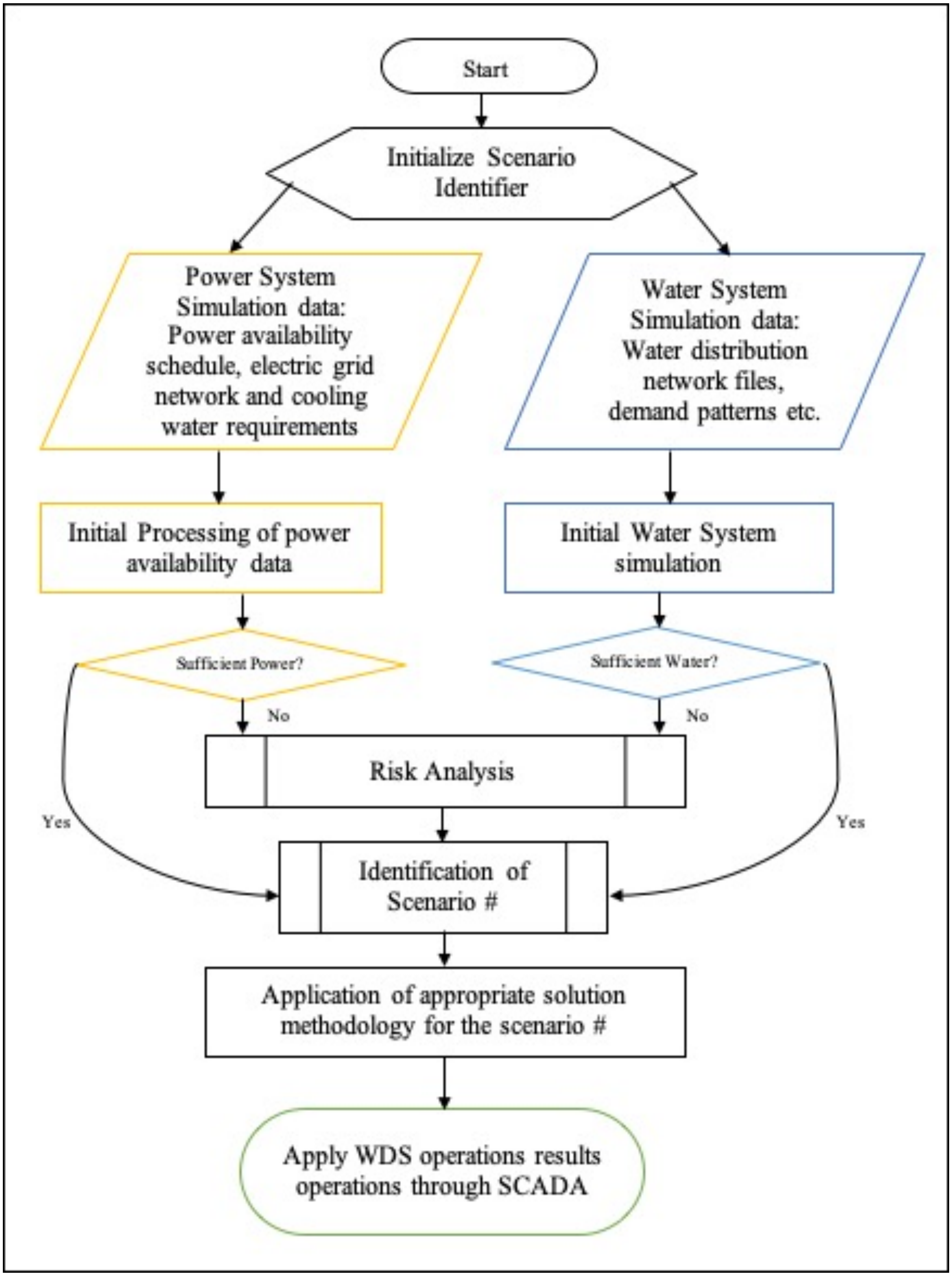


Figure 9-1 Adaptive Management Methodology through Targeted Intervention for

Application of Water – Energy Nexus Operations Model

As a part of this research, an adaptive management strategy was introduced toachieve a lucid and practicable implementation of the technological solution developed in CRISP type 2 project. Figure 9-1 gives a system diagram for implementation of the energy - water nexus optimization – simulation model through a process of risk analysis and targeted intervention to the operations of the nexus. Several scenarios were developed based on forecasts and predictions of the various conditions affecting the normal operations of the nexus. These scenarios were developed based on historic observations and future predictions of the various mutual interdependencies between the power and water systems and a scenario number is assigned for each. A methodology for performing a risk analysis for real time situation based on the data received from the water SCADA system was developed in addition to a method for scenario number identification. The scenario identification algorithm is initiated when the data is received from the power and WDS SCADA systems. As shown in Figure 9-1, initial simulations of both the water and power systems are performed to receive the initial predictions of the various parameters for an extended future time-period. After this step, it is determined whether sufficient power and water is available for smooth functioning of both the distribution systems to facilitate both the systems to perform their respective valued functions. A risk analysis is then performed if a power or water deficiency is observed and a scenario number is identified for that particular period of time. This process is then followed by implementation of the appropriate solution methodology for the scenario identified. The results of these implementation are then implemented for the operations of the WDS and the power system for the next time-period on a real-time basis.

9.4. Application of Earth Systems Engineering and Management (ESEM) Principles for Managing Risks and Improving Resilience of the WDS Operations

An intricate administration and communication system would be required for ensuring a transparent governance for achieving a dialogue between the various stakeholders. Though transparent governance conventionally involves a democratic system, a democratic system for critical infrastructures is only practicable for long term policy decisions rather than the short term instantaneous decisions required to be made during emergency situations and contingencies. It is proposed that reasonable multicultural dialogue in the form of various consumer surveys and public meetings would be performed on a regular basis toachieve public acceptance and social legitimacy for any long-term policy decisions with regards to WDS operations. The decisions which could be achieved through public consent include fixing times of reduced water supply for each zone of the city in case of a drought, increased water tariffs during peak hours etc. The techno-social differentiation principle of ESEM would be achieved in this project by using social engineering including a two-way communication with the various stakeholders as explained earlier for policy decisions without any social interference in the technical functioning or operations of the system. Thus, the characteristics and functioning of the system and its artifacts including the water agency, consumers, governments etc. are considered while developing the methodology for the overall control.

To summarize, this project envisages the ESEM principles as follows:

- **Targeted intervention** – Contingency protocols and trigger points

- **Evaluate technological fix** - application of methodology for simulation of emergency scenarios, extreme drought conditions etc.
- **Real – World Boundaries** – Non-consistency in designed systems across the globe, uncertainties in climate models, **no explicit control** or central control for the overall water – energy nexus, social boundaries, etc.
- **Multi-dimensional dialogue –** Emergency communications with stakeholders using television and radio broadcasts, public addressing systems etc. Inclusive policy making mechanism using surveys, public meetings etc. for long term decisions.
- **Techno-social differentiation –** The social aspect of this project would deal with the policy decisions while the technical aspect would deal with the detailed functioning and operations of the nexus. Thus, a clear differentiation would be achieved.
- **Transparent governance -** An intricate administration and communication system would be required for ensuring a transparent governance. A democratic system for critical infrastructures is only practicable for long term policy decisions.
- **Multicultural dialogue -** Reasonable multicultural dialogue in the form of various consumer surveys and public meetings would be performed on a regular basis toachieve public acceptance and social legitimacy.
- **Part of the system -** The technical solution presented here developed as a part of the system rather than an externality.

- **Systems and artifacts** - The characteristics and functioning of the system and its artifacts including the water agency, consumers, governments etc. are considered while developing the methodology for the overall control.
- **Continuous learning -** The methodology considers a continuous learning approach wherein its application could be easily modified for any new data available at a future time.
- **Long –term investments -** Along with the short-term operation changes proposed by use of the mathematical models, a few system improvements would also be required as a long term investment.
- **Quantitative metrics -** demand satisfaction (ratio of demands satisfied to the required demand), pressure constraint violations, power constraint violations, consumer satisfaction, etc.
- **No Explicit Control -** real world boundary for the water – energy nexus is that there is **no** central control or explicit control over the entire system, but a network of several control points, which are to be managed implicitly through a middleware.
- **Expect emergence –** The methodology considers unexpected population growths leading to unpredicted increases in demands in the various parts of the system. It is also designed not to be dependent on the artificial boundaries and thus accepts any unforeseen expansion of the WDS within reasonable natural boundaries.

- **Incremental and reversible -** The quantitative metrics used here to ascertain system performance are instrumental in ensuring an **incremental and reversible** alteration to the WDS.

9.5. Using Adaptive Management for Sustainable Operations of WDS

The adaptive management strategy developed for optimization and control of water distribution systems (WDS) in this study, aims at operating the WDS toallow it to fulfil its valued functions such as supplying water to all the consumer nodes in the system, maintenance of a pressurized flow in the system, efficient use of reclaimed water, prioritization of various demands within the system etc. even during extreme emergent conditions like short term and long term limited energy availability, extreme droughts, natural disasters, situations arising out of foreseen and unforeseen population growth, future water policies, climate change etc. Brundtland (1987) in the Report of the World Commission on environment and development (WCED) define sustainability as “the development that meets the needs of the present without compromising the ability of the future generations to meet their own needs.” While discussing the concept of sustainability, Allenby (2014) explains that the definition given by WCED was the first and remains the most authoritative. This being said, it can be reasonably argued that the WCED definition is generalized and considers just the egalitarian point of view rather than providing a more comprehensible and practicable solution, which is essential for solving any engineering problem like the one considered here. Considering just the WCED definition, the optimization – control methodology developed as a part of this project fits perfectly well in the sustainability discourse. Firstly, the adaptive management strategy developed herein considers several scenarios which involve various degrees of system disruption because ofthe different factors ranging from short term and long term effects of natural disasters to failure of components of the two systems being considered. A targeted intervention is achieved here toachieve robust and reliable operations of the water – energy nexus as a whole. Secondly, the methodology allows an efficient and effective use of the critical infrastructures in the urban areas, which falls in line with the sustainable discourse as explained by Allenby (2014). Not only does the adaptive management strategy developed in this project addresses the needs of a reliable WDS for the present, but also for predictions of future use trends and conditions.

Hazard as defined by Allenby (2014) as a pathway or a possibility for a certain adverse event occurs within a system. These pathways could be compared with the different scenarios considered in the research for a contingency in the water system arising due to the effects of various interdependencies of the water – energy nexus being considered in the study. Rogers et al. (2002) discusses the theory of treating water as an economic good for an improved sustainability in the water distribution systems. Treating water as an economic good lead to a conundrum viz. unequal distribution of water leading to conflicts within a society. The sustainability approach for water distribution and management suggested by Rogers et al. (2002) considering water merely as an economic good is thus rather flawed as far as the WCED definition of sustainability is considered, since it does not lead to an egalitarian society. On the other hand, water infrastructure being of critical infrastructure, requires a high amount of public and private investment, which in a real world scenario requires it to be considered as an 'economic good'. Hussey and Pittock (2012) provide a rather different view for achieving sustainability in water – energy nexus operations by an approach of resource management and equal distribution. The methodology of adaptive management considers a middle path in this case, wherein water is considered as both a right as well as an economic good. In case of normal (non- emergency) operations of the water distribution system, the optimization models used as a part of the overall methodology attempt to optimize the operations of the WDS based on power consumption and resource management objectives, leading to efficient and economic operations of the WDS and the power generation. On the other hand, when the water – energy nexus encounters an emergency situation as shown in the various scenarios formulated in this project, a targeted intervention is achieved through application of the appropriate model as explained in Table 9-1. This targeted intervention minimizes the deficit in water supplied to various consumers based on their priority of use rather than economic considerations in an emergency short term situation or a long term contingency occurring in the nexus.

Mays (2007) provides a detailed discussion and several methodologies for achieving sustainability in operations of the water distribution systems and water infrastructure while providing several examples of such sustainable water systems from the ancient world. Mays (2007) addresses several issues in his book regarding water resources sustainability including hydraulic, water quality, public health, WDS security, reliability, resilience and economic considerations. Along with these several issues affecting the sustainability a major consideration for achieving sustainable operations of the WDS would encompass water – energy systems interdependencies (nexus). Figure 9-2 gives the various facets for sustainable WDS operations. The methodology developed in this project deals with all these facets of sustainable operations of the WDS and thus can be sufficiently portrayed as an instrument for achieving the overall sustainability thereof.

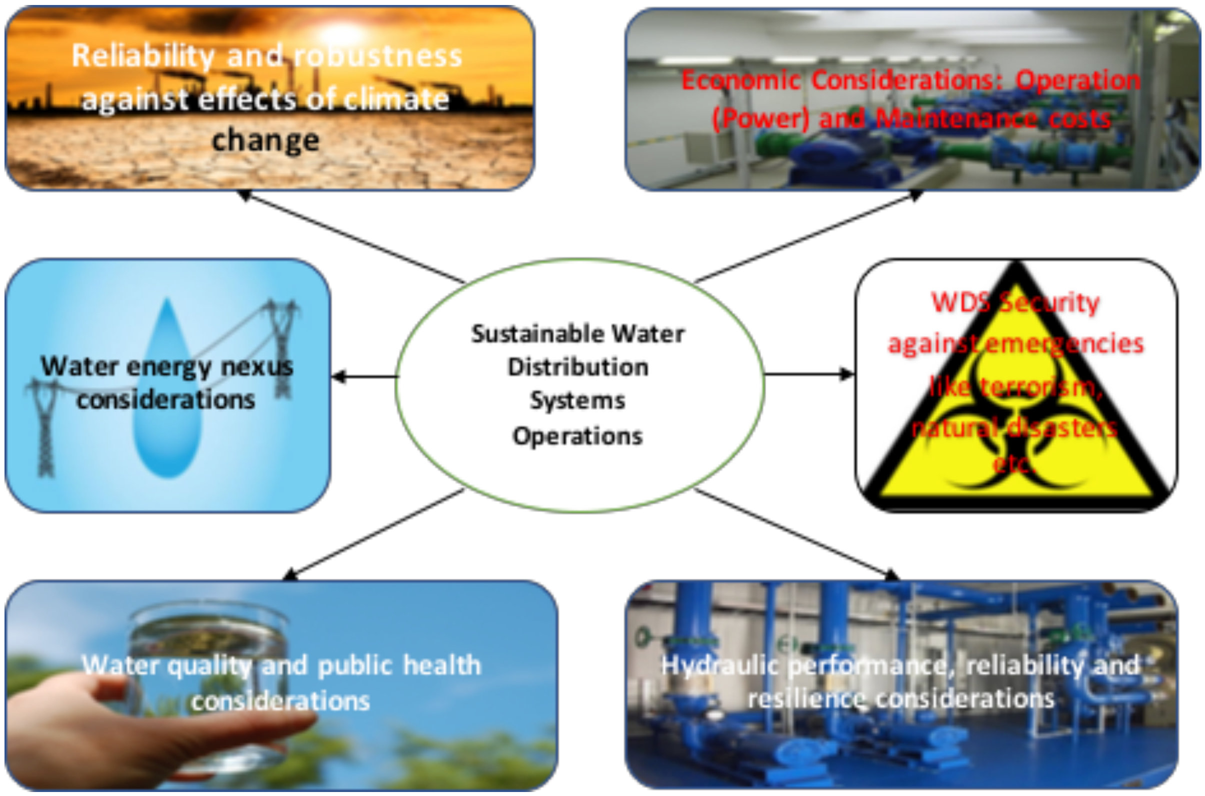


Figure 9-2 Sustainable Water Distribution Systems Operations

9.6. Technological Implications of the Proposed Methodology

The technological solution achieved through the methodology developed in this study may be a part of the cluster surrounding the water – energy nexus management technologies as a part of the overall water distribution management technologies and policies. Though the two systems viz. water distribution systems and the electricity grid have existed in most areas of the United States for over a couple of centuries now, a wave of combined analyses and development of the two considering the various interdependencies is yet to come. The various white papers and reports published by

United States Government Accountability Office (2011, 2012a., 2012b.); Committee on Energy and Natural Resources, United States Senate and Congress (2014) and Copeland et al. (2014) address the need of urgent policy making and technological development for water – energy cluster as a combined and integrated system rather than independent developments. The water – energy nexus technological cluster has opened several avenues of development and policy making and the methodology presented herein provides a tool for decision making for many of the foreseen scenarios that may lead to a shift in the policies related to the nexus management. The technological solution developed was in part driven by an unprecedented development in the field of water distribution supervisory control and data acquisition (SCADA) systems during the last few decades as well as the various policies developed over the years for management of the water distribution systems. As far as the impacts of this technological innovation are considered, it would be rather premature at this stage to forecast the wide range of impacts it could have. One of the major impacts include improvement of the SCADA systems to provide controls at the end user level, to facilitate more efficient water management. Other than that, the complexities of influence are rather veiled than obvious at the current stage.

The technological impacts of this methodology could be assessed by considering the technological impact framework given by Allenby (2014) considering the various levels of impact on the overall socio – economic scenario. The level I or immediate impacts of this methodology would be an improved resilience and reliability of the water – energy nexus. The methodology for the first time provides a logical decision support tool for demand satisfaction during emergencies. This would enable the related agencies to formulate policies regarding operation of these critical systems under such scenarios leading to a more sustainable urban infrastructure management. Level II or associated impacts include the economic advantages of consideration of water – energy nexus rather than considering them as separate systems. A revision in the pricing and billing systems in practice considering the water and power demands in combination could prove profitable to both the water as well as power industries. Level III impacts which involve 'earth systems' or social impacts of the system are more difficult to assess. A few easily evident impacts of application of this methodology include changes in the trends of water and power use based on a combined pricing structure, better compliance with water rights leading to a more sustainable society, solution of water conflicts through amiable and logical solution etc.

10. SUMMARY, CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE RESEARCH

10.1. Summary and Conclusions

A water supply system (WSS) conveys water from sources (such as lakes, rivers, dams etc.) to the treatment plants and then to users via the water distribution systems (WDS) and/or water supply canal systems (WSCS). New methodologies for optimal control of water supply systems (WSS) under conditions of limited water and/or power availability are developed and tested for both hypothetical and real example water distribution systems (WDS) and water supply canal systems (WSCS), with results depicting improved resilience for operations of the WSS under normal and emergency conditions. The methodologies presented could be used for real-time control as well as short and long-term contingency planning for WDS and WSCS operations. The results of the example applications show that the optimization-simulation models could be used for regional WSS as well as for city WDS at a higher resolution. A new concept of infrastructural – operational resilience (IOR) is presented as a comprehensive WDS performance assessment tool.

10.1.1. WDS Pump Operations under Demand Uncertainties

A methodology for generation of optimized pump schedules for certain and uncertain demands is presented in Chapter 3. This model, if implemented for an urban water distribution pumping system could result in considerable reduction in water supply expenses. The method can be applied to operations of a medium to large-scale municipal water distribution pumping station. Such an application of the methodology provides an optimized pump schedule for the system. The various requirements for the water distribution network such as tank levels and pressure requirements are taken into consideration in the head constraints. This provides methodology for real-time control and decision making for pumping facilities of small to large-scale water distribution systems while considering the uncertainty in demand satisfaction, which is inherent in any real water distribution system.

10.1.2. Real-time WDS Pump Operations under Conditions of Limited Power and/or Water Availability

The model presented in Chapter 4 is a novel approach for optimization of real-time WDS pump operations under normal as well as limited power conditions. In addition to minimizing the difference between required and satisfied demands, the model also achieves compliance of the water quality requirements for total dissolved solids (TDS) at the power plants. The model employs a methodology for minimizing the difference between the demands requested and the demands satisfied to optimize pump and valve operations of the WDS.

A methodology for control of pumps and valves using an optimization-simulation approach is presented in Chapter 5. The algorithm presented is a novel methodology for determination of optimal operations for water distribution systems under normal and emergency conditions of limited power availability and/or water availability. The model employs a methodology for minimizing the difference between the demands requested and the demands satisfied to optimize pump and valve operations of the WDS. The example application of the model provides evidence of applicability of the model for real-time operations of a WDS under limited power availability. The demands are observed to be satisfied in the period of sufficient power availability while the demand satisfaction is curtailed in case of limited power availability time periods. Improved demand satisfaction was observed with consideration of both pump and valve controls.

Storage of finished drinking water in tanks leads to the degradation of water quality through chemical, physical and biological processes that occur as water ages and through external contamination of water in tanks. Water quality problems associated with storage of finished water in tanks include loss of disinfectant residual, formation of disinfection byproducts, development of flavors and odors, increase in pH, corrosion, buildup of iron and manganese, and the occurrence of hydrogen sulfide and leachate from internal coatings. An implicit objective in both the design and operation of distribution system storage facilities is the minimization of detention time and the avoidance of parcels of water that remain in the storage facility for long periods. Tank turnover is the timely replacement of water stored in a tank through consumptive use and pumping. Timely turnover of the water stored in a tank leads to a reduction of water age and improvement in the water quality. Thus, tank turnover rates are an important requirement for efficient operation of a water distribution system (WDS). Water age in storage tanks can be managed through routine turnover through fluctuations in the water levels. A model for optimization of daily pump operation schedules and valve controls considering the tank turnover requirements for a WDS is presented in Chapter 6. In addition to minimizing the power costs associated with pumping required in a WDS, the model also achieves compliance of the system operations with tank turnover requirements by optimizing the pump and valve operations.

10.1.3. Performance Assessment of WDS Operations under Conditions of Limited Power and/or Water Availability

Performance of water distribution systems under the conditions of limited power and water availability is an important consideration for design and operation of a water smart city. The aim of this study is to present a mathematical formulation for system performance resilience under considerations of limited water and power availability conditions. A new methodology for determining system operation resilience is presented for the real-time operation of water distribution systems (WDS) under critical conditions of limited water and/or limited electrical energy resulting from extreme drought or electric grid failure. Resilience for water distribution systems is defined as how quickly the WDS recovers or bounces back from emergency operations to normal operations. The algorithm for operational resilience is interfaced with an optimization-simulation model for the real-time optimal operation of water distribution systems. The resilience methodology considers both demand and water quality requirements of both the municipal WDS and the power plant cooling systems. An infrastructural – operational resilience (IOR) computational method presented for the inclusion of infrastructural robustness calculations within an extended time-period WDS optimization-simulation framework (explained in chapter 4). The IOR methodology could be applied to forecasted, extreme, mega-drought scenarios for more accurate system simulations and anticipatory policy making.

10.2. Future Research Recommendations based on Water – Energy Nexus Considerations

10.2.1. Water Supply System Control Methodology Based on Water-Energy-Food Nexus Considerations

Introduction

Water, food and energy are three critical resources that are essential for sustenance/development of any community. Independent network systems are used for generation/creation and distribution of these three resources. These highly interdependent systems form an intricate nexus termed as 'water – energy – food nexus'. The main dependency of the water supply system (WSS) on the power distribution system (PDS) is the electrical power required for pumping, while the main dependency of the PDS on WSS is the cooling water required for operation of power plants. Food system is a collective term for agriculture, food preservation/ storage and distribution systems. Food systems are dependent on WSS for irrigation water and energy requirements as well as energy required for refrigeration, processing and distribution of food. Because of the interdependencies of these three systems, a systems approach would be required for modeling and optimization of operations during contingencies such as droughts, electrical grid failure, crop failures, etc. The methodologies presented in this research address the water and energy systems. The proposed future research would involve extension of the methodologies presented in this study to include modeling and operations of the food systems and their interdependencies on WSS and PDS. An intricate administration and communication system would be required for ensuring an effective dialogue between the various stakeholders for efficient combined control of water, energy and food systems.

Problem Statement

Agriculture is a major user of ground and surface water in the United States, accounting for approximately 80 percent of the Nation's consumptive water use and over 90 percent in many Western States (Kassel, 2013). On the other hand, The U.S. agriculture industry used nearly 800 trillion British thermal units (Btu) of energy in 2012, or about as much primary energy as the entire state of Utah. Thus, the food system is a critical infrastructure that is highly dependent on both WSS and PDS. The purpose of this proposed future research is to extend the water – energy nexus combined control methodology discussed in this study, to include the food system facet of the intricate water – energy – food nexus system. The modeling effort to be undertaken would include development of a systems approach to address all the interdependencies for operating the three systems.

Objectives

Specific questions to be addressed by the proposed future research are:

i. What are the specific interdependencies and bottle necks between the water, energy and food systems?
ii. What are the natural and manmade contingencies and constraints in operations of the three systems?
iii. What is the best way to allocate water during emergency/contingency situations?
iv. What is the best way to operate the three systems during contingencies?
v. For long-term planning, what should be the cropping pattern and irrigation water allocation for a particular during limited water and/or power availability?

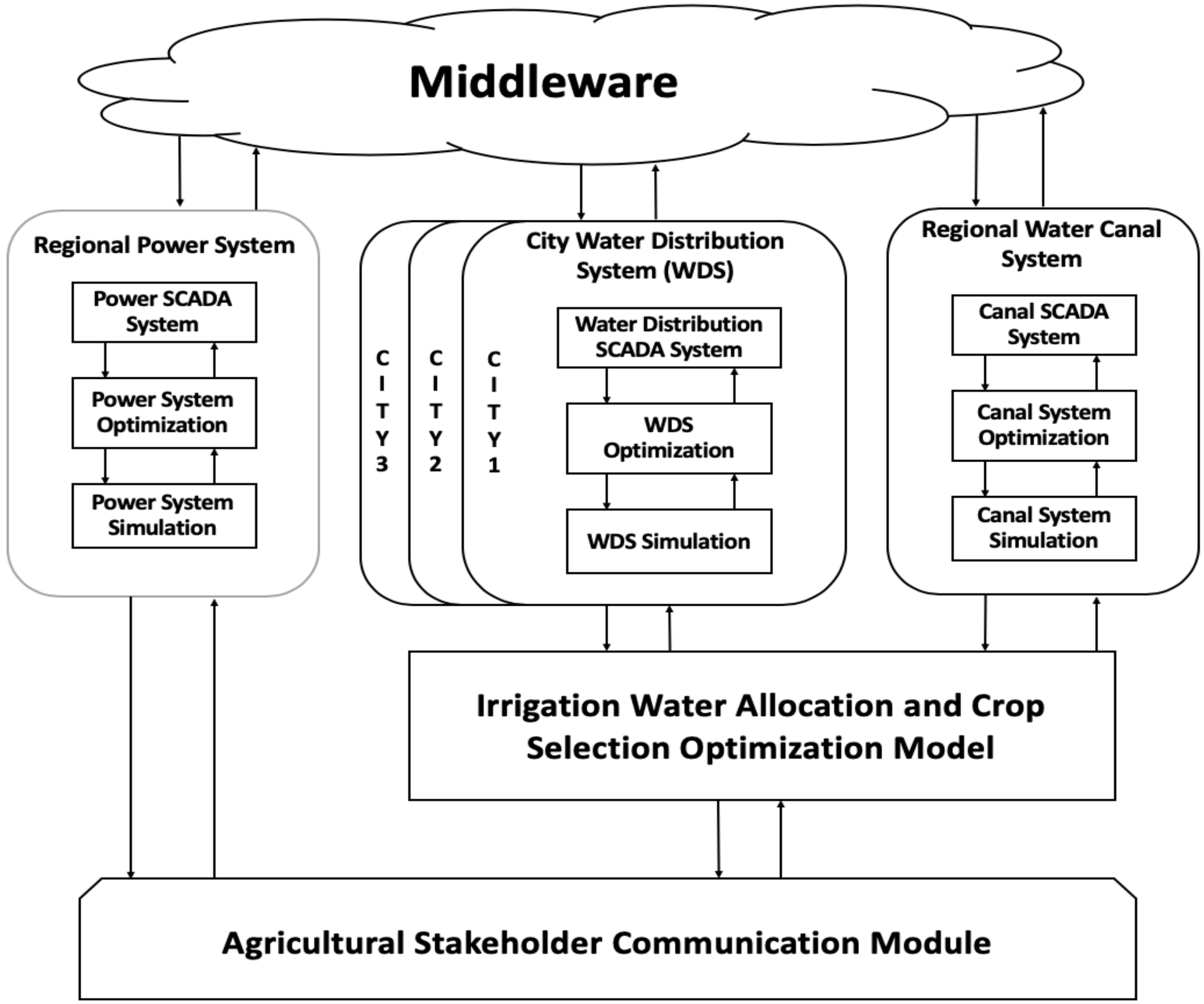


Figure 10-1 Proposed Methodology for Water - Energy - Food Nexus Operations

Proposed Methodology

The future research would include interfacing of an optimization model for irrigation water allocation such as model given by Aljanabi et al. (2018). A model for selection of crops based on water and energy availability would also be developed and interfaced. This would provide a short term as well as long term planning tool for water allocation and crop selection for the regional agriculture based on water – energy – food nexus considerations. Figure 10-1 shows a conceptual schematic of the proposed methodology for water – energy – food nexus operations. The optimization models for water allocation and crop selection would aim at maximization of food production in a certain region while keeping the irrigation water and energy usage within the availability.

10.2.2. Combined WDS – WSCS Infrastructural – Operational Resilience (IOR) Computation Methodology under Conditions of Limited Power and/or Water Availability

Introduction

Water supply canal systems (WSCS) are used for conveying water from a source such as a dam or a river to the water treatment plants for urban water supply or to the agricultural fields for irrigation. WSCS can consist of a single canal or a more complex system with several diverging canals that deliver water to various demand locations. WSCS can be large consumers of energy for pumping especially at locations where the topography of the region mandates lifting of water. WSCS are critical infrastructure systems and are an important part of the regional water supply system (WSS). Operations of WDS and WSCS are highly interdependent. For a comprehensive performance assessment of regional WSS operations, a combined resilience computation methodology for WSCS and WDS infrastructure would be required.

Problem Statement

The proposed research would aim at development of a combined infrastructural – operational resilience computation methodology based on the IOR concepts defined in Chapter 7. New infrastructural robustness metrics and measure of performance would be defined for WSCS operations. The proposed research would include defining operational resilience and infrastructural resilience for each measure of performance for WSCS. Combined IOR for WDS and WSCS operations under conditions of limited water and/or power availability would be defined based on a weighted average of WDS IOR defined in Chapter 7 and the IOR for WSCS systems.

Objectives

The objective of the proposed research is to develop a computational methodology for infrastructural – operational resilience (IOR) for combined operations of WDS and WSCS during conditions of limited power and/or water availability.

Specific questions to be addressed by the proposed future research are:

i. What are the measures of performance for WSCS operations?
ii. How to quantify the interdependencies between water distribution systems (WDS) and water supply canal systems (WSCS)?
iii. What factors affect the infrastructural robustness of WSCS systems?

Proposed Methodology

Concepts of operational resilience and infrastructural robustness are defined in Chapter 7 along with a new methodology for computing infrastructural – operational resilience (IOR) for water distribution systems (WDS) under conditions of limited water and/or power availability. Future research would aim at expanding this methodology to include WSCS. The various measure of performance for operations of WSCS under conditions of limited power and /or water availability include water surface elevation (WSE) set-points, pump operations including switch constraint violations and power availability violations and demand satisfaction. Infrastructural robustness parameters to be considered for WSCS include betweenness, connectivity and water demand priority. A combined WSS IOR (as shown in Figure 10-2) would be defined using a weighted mean of WDS IOR and WSCS IOR. This methodology would provide a tool for utilities to assess the overall performance of the regional WSS and plan future operations.

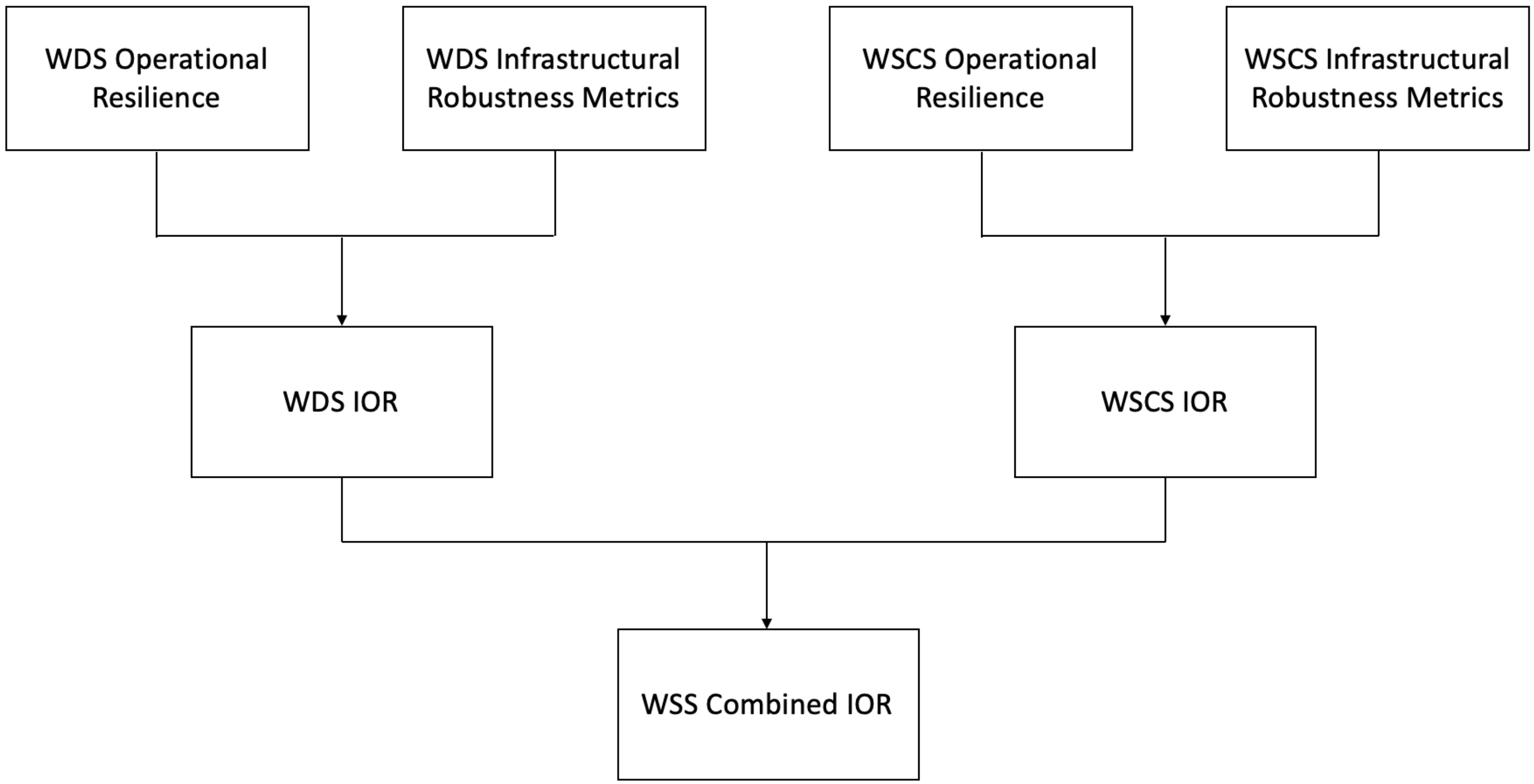


Figure 10-2 WSS combined IOR computation methodology

10.3. Future Research Recommendations for Sustainable WSS Operations

10.3.1. Optimization-Simulation Model for Adaptive Operation of Water Supply Systems under Extreme Weather Conditions

Introduction

Extreme weather events such as floods and droughts are increasing in frequency and intensity on account of global climate change. These events adversely affect the operation of water supply infrastructure both in short and long term. Simulation of the interdependencies between water and energy infrastructures and optimization of the operations of water supply systems (WSS) for short and long-term limited water and/or energy availability have been discussed in Chapters 3 – 7. Pellicer et al. (2013) advocate use of adaptive and sustainable management for WSS of future water smart cities. Adaptive management of WSS operations for extreme weather events includes crisis management in short term and preparedness through system level changes for the long-term (Sinisi et al., 2011). Current research achieves the crisis management through real-time optimal control of WSS. Interfacing of a numerical weather prediction system (NWPS) with the optimization-simulation models for optimal control of WSS, would provide for a long-term planning tool for future extreme weather events. Adaptive system changes for WSS would include changes in pump and valve operations for WDS and gate and pump operations for WSCS.

Problem Statement

Climate change is a major global challenge for the human race. Critical water infrastructure systems such as water distribution systems (WDS) and water supply canal systems (WSCS) are under tremendous pressure on account of their design capacities being exceeded by the ever-growing demands and the dwindling availability of water. The problem is further worsened by the extreme weather conditions caused by the ever-changing climatic conditions. This proposed research aims at developing a methodology for optimal operations of WSS under extreme weather conditions like droughts and floods. The proposed methodology would be a tool for short term as well as long term adaptive management of WSS.

Objectives

The objective of the proposed research is to develop a combined optimization-simulation methodology coupled with NWPS for optimal control of WSS (WDS and WSCS systems) under future extreme weather conditions.

Specific questions to be addressed by the proposed future research are:

i. What are the foreseen implications of future extreme weather conditions on operations of WSS?
ii. What is the most sustainable way to operate pumps, valves and canal gates in WSS under such extreme weather conditions?
iii. How resilient are the system operations for future extreme weather conditions?

Proposed Methodology

Methodologies for optimal control of WDS and WSCS under conditions of limited water availability and/or power availability have been presented in Chapters 4 and 8 respectively. The proposed future research would include interfacing these methodologies with a numerical weather prediction system (NWPS). Figure 10-3 shows the schematic of the proposed methodology for adaptive operation of WSS under extreme weather conditions. The predicted future weather data from the NWPS would be used as an input for a regional hydrological model (developed in a hydrological simulator such as HEC-HMS). The data includes precipitation, duration of precipitation and temperatures in the region. The output from the regional hydrological model including water surface elevations (WSE), water availability and reservoir/ storage facility precipitation inflows would be thereafter sent to the optimization-simulation models for WDS and WSCS respectively through a centralized control system for WSS operations. The optimization of controls would be performed through the respective models and a supervisory control and data acquisition (SCADA) system would thereafter be used for actually controlling the pumps, valves and gates in the WSS at real-time.

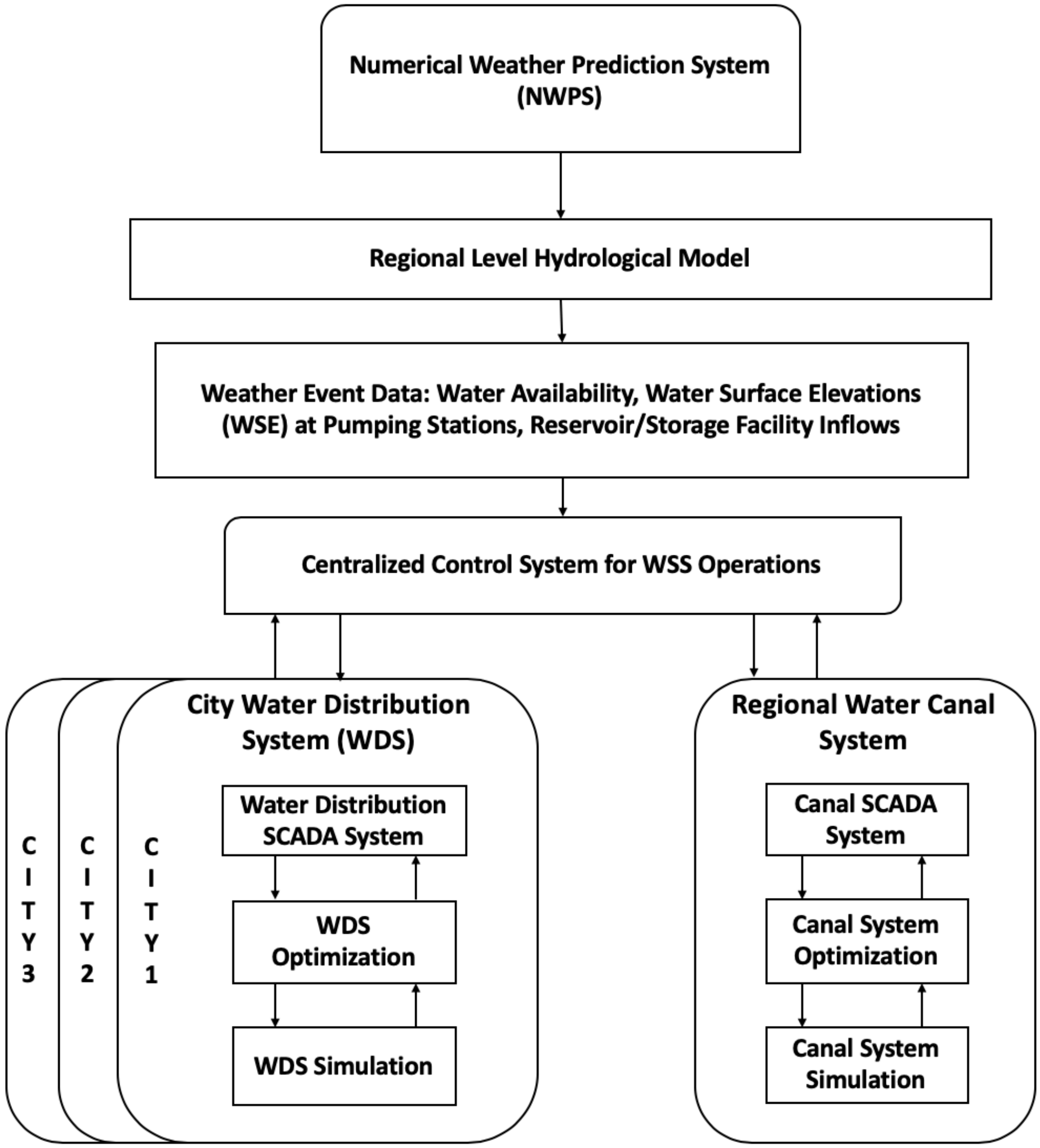


Figure 10-3 Adaptive operation of WSS under extreme weather conditions

10.3.2. Testing an Optimization – Simulation Model for Water Distribution System Pump and Valve Control Considering Operations of Hydropneumatic Tanks

Introduction

Hydropneumatic tanks (HT) are defined by Dodd (1943) as vessels that hold water and air under pressure in order to provide efficient water supply while regulating pressures in the water distribution system (WDS). Several WDS are equipped with HT, for providing pressurized water quickly and on-demand, without requiring constant pumping. Use of HT makes a WDS more efficient and adaptive, since for a small increase in water demand, additional pumps are not required to be used. In addition, these tanks can be used in conjunction with booster pumps to deliver water when the system is in a period of a short-term shutdown. Figure 10-4 shows the various components of hydropneumetic tank. Operation of a hydropneumatic tank includes operation of pumps and valves in the system required to maintain the pressures in WDS and the required tank levels within the hydropneumatic and water storage tanks. An optimization-simulation model is presented in Chapter 6, for determining the WDS pump and valve operation schedules that minimize the power costs or energy used for pump operations and satisfy demands, pressure, and tank turnover requirements. The proposed research would aim at extending the application of this optimization-simulation model for WDS equipped with HT.

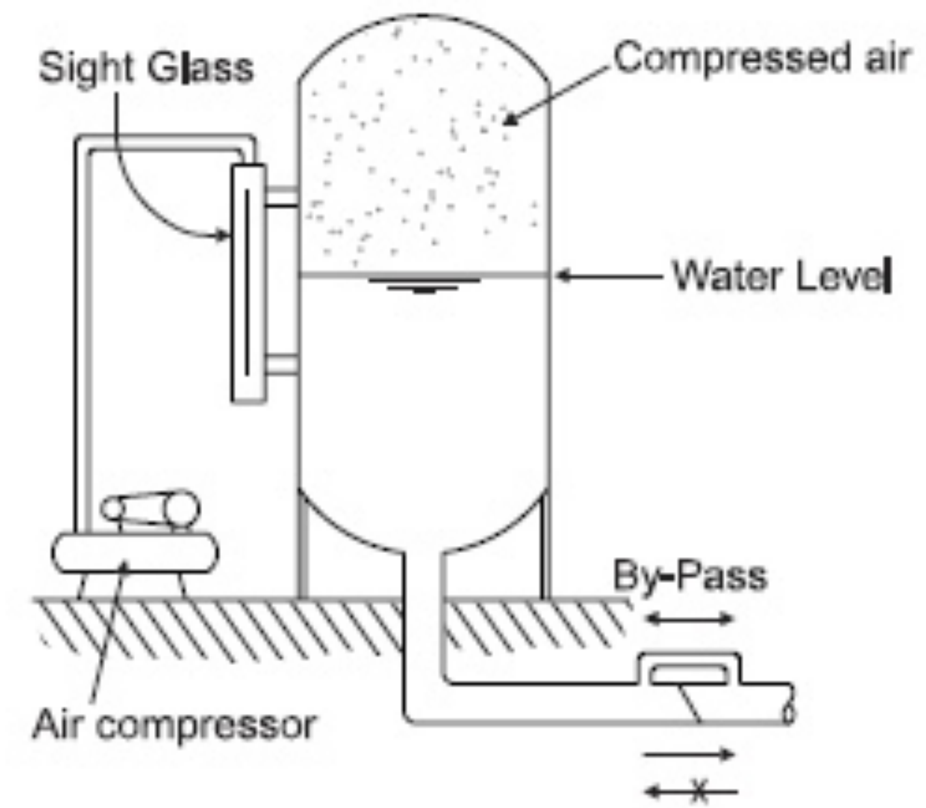


Figure 10-4 Components of a Hydropneumatic Tank (Source: Bentley)

Objectives

The aim of this research is to test the optimization – simulation model presented in Chapter 6 for application to a WDS equipped with hydropneumatic tanks. The specific questions to be answered by this research include:

i. What is the most effective way to model a hydropneumatic tank for an extended period simulation?

ii. What is the optimal way to operate the pumps and valves for a WDS equipped with a hydropneumatic tank?

iii. How effective are the hydropneumatic tanks in the WDS for meeting the consumer demands and maintaining the pressures within acceptable limits in the system during normal flow conditions and fire flows?

Proposed Methodology

The methodology for the proposed research would include formulation of a mathematical optimization model for operations of the WDS with hydropneumatic tank considerations and solving the optimization model through an interface between the optimization model (Genetic algorithm in MATLAB) and the hydraulic and water quality simulator such as EPANET (Rossman, 2000) for reaching an optimal solution through iterations between the optimization and simulation models. For the purpose of this application, the optimization – simulation model for WDS operations presented in Chapter 6, would be extended to include constraints related to hydropneumatic tank operations. Figure 10-5 shows an implementation overview for the proposed optimization-simulation model.

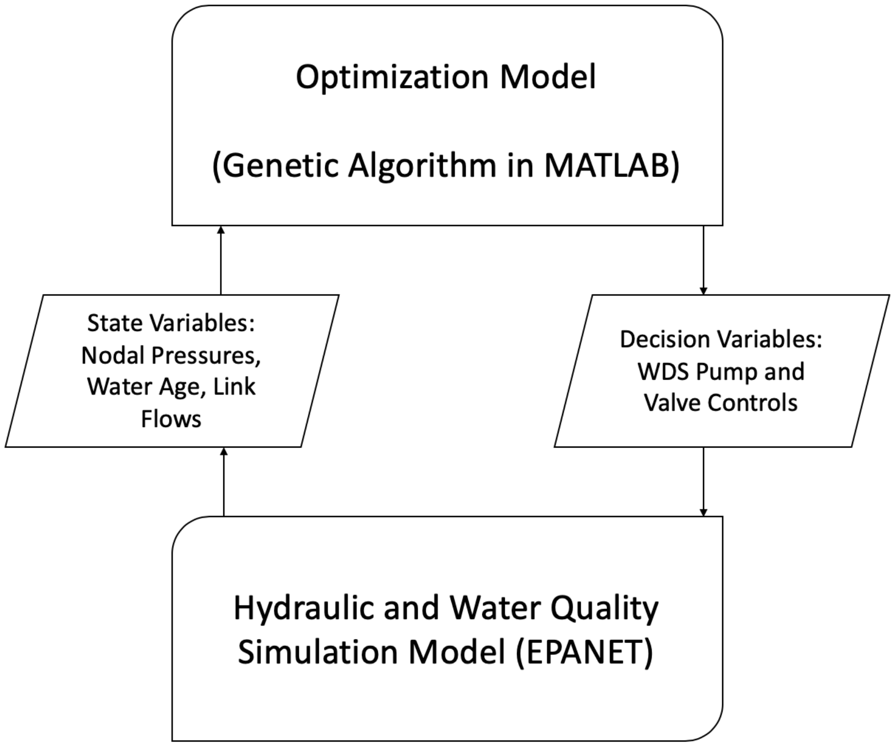


Figure 10-5 WDS Optimization – Simulation Model Considering Hydropneumatic Tanks

In most tanks, the water surface elevation (WSE) in the tank equals the hydraulic grade line (HGL) in the tank. In the case of a HT, however, the HGL is higher than the WSE, since HT are partly full of compressed air. Walski et al. (2003) give the following expression for the HGL of a HT

$$\mathrm{HGL} = \mathrm{C_f P} + \mathrm{Z} \tag{10-1}$$

Where, HGL is the elevation of hydraulic grade line for the HT, $\mathrm{C_f}$ is unit conversion factor (2.31 English, 0.102 SI), P is the pressure recorded in tank (psi, kPa) and Z is the elevation of tank (ft, m).

For an extended-period simulation in EPANET, HT is represented by an equivalent free-surface tank floating on the system. Because of the air in the tank, a hydropneumatics tank has an effective volume that is less than 30 to 50 percent of the total volume of the tank (Walski et al., 2003). Modeling the tank includes first determining the upper and lower bounds on the pressures to be maintained within the tank and then converting them into HGL using equation 10-1. The cross-sectional area (or diameter) of this equivalent tank can be then determined using the following equation given by Walski et al. (2003).

$$A_{eq} = \frac{V_{eff}}{\overline{HGL} - \underline{HGL}} \tag{10-2}$$

Where, A_{eq} is the area of equivalent free-surface tank, V_{eff} is the effective volume of the HT, $\overline{HGL}$ and $\underline{HGL}$ are the hydraulic grade line elevations corresponding to the upper and lower bounds respectively of pressure within the HT. $\overline{HGL}$ and $\underline{HGL}$ would be modeled as upper and lower tank level setpoints within the optimization-simulation model.

Preliminary Mathematical Formulation

An optimization-simulation model is given in equation (6-1) – (6-10) for optimal pump and valve controls considering tank turnover requirements. For extending the optimization model to model for HT, the following bound constraint would be required to be added for every HT in the WDS in addition to the constraints given in equations (6-2) – (6-10).

$$\underline{HGL} \leq y_{s,t} \geq \overline{HGL} \qquad \forall\ s = 1,...,S \text{ and } t = 1,...,T \tag{10-3}$$

Where, $y_{s,t}$ is the water level in tank s at time t.

A reduced optimization model with constraints in the form of penalty functions is solved by the genetic algorithm, which solves unconstrained problems. Equation 6-11 is reformulated to include HT constraint given in equation 10-3. The reduced optimization model for WDS pump and valve controls considering hydropneumatics tanks (HT) is given as follows:

$$\text{Minimize } \mathrm{Obj}_{\mathrm{reduced}} = W_1 \sum_{p=1}^{p=Pu} \sum_{t=1}^{t=T} \left[X_{p,t}\, \emptyset_t \frac{Q_{p,t}\, H_{p,t}}{3956\eta_{p,t}} \right] +$$

$$W_2 \sum_{k=1}^{k=K} \sum_{t=1}^{t=T} \left[w_1 \max(0, 0 - P_{k,t}) + w_2 \max(0, \underline{P} - P_{k,t}) + w_3 \max(0, P_{k,t} - \overline{P}) \right] +$$

$$W_3 \sum_{s=1}^{s=S} \left[\sum_1^{T/24} \max\left(0, \overline{y_{\mathrm{req}\,(s)}} - \max([y_{s,t}]_{t=0:T-24}^{t=24:T})\right) + \right.$$

$$\left. \max\left(0, \min([y_{s,t}]_{t=0:T-24}^{t=24:T}) - \underline{y_{\mathrm{req}\,(s)}}\right) \right] +$$

$$W_4 \sum_{t=1}^{t=T} \sum_{s=1}^{s=S} \max\left(0, \left[\theta \max\left(\left[\mathrm{Vol}_{s,t} \right]_t^T \right) - \sum_t^{t+24} X_{\mathrm{out}\,(s,t)} Q_{\mathrm{out}\,(s,t)} \right] \right) +$$

$$W_5 \sum_{s=1}^{s=K} \sum_{t=1}^{t=T} \left[w_2 \max(0, \underline{\mathrm{HGL}} - y_{s,t}) + w_3 \max(0, y_{s,t} - \overline{\mathrm{HGL}}) \right]$$

(10-4)

Where, $\mathrm{Obj}_{\mathrm{reduced}}$ is the reduced objective function, W_1 to W_5 are penalty weights associated with objective function, pressure bounds, tank level bounds and tank volume turnover constraints respectively, and w_1 to w_3 are additional penalty weights for negative pressures, lower pressure bound violations and upper pressure bound violations respectively.

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APPENDIX 1

AMPL CODE FOR OPTIMIZATION MODEL IN CHAPTER 3

```
#set and parameters:

param npump >=0;                  # Number of Pumps
set pump:= 1..npump;                 # set of pumps
param time>=0;                    # time of simualtion
set tim:= 1..time ;                # set of times
param Qd;             #Average total demand for the system
param pat{t in tim};    #Demand pattern for the system

param g;              # acceleration due to gravity
param f_eq;             # friction factor for Eq. pipe
param L_eq;            #lengh of Eq. pipe
param D_eq;            #diameter of Eq. pipe

param Y_tank;           #required level in tank
param ElevT;                    #Average elevation of storage tanks from the ground level.

param H_suc;            # suction height
param f_suc;            # friction factor for suction
param L_suc;            #lengh of suction pipe
param D_suc;            #diameter of suction pipe

param f_del;             # friction factor for delivery pipe
param L_del;             #lengh of delivery pipe
param D_del;             #diameter of delivery pipe
param ElevN;             # Avg. elevations of all nodes
param hmin;                               # min head at node
param hmax;                              # max head at node


param Z_pump;                                  # elevation of pump
param Pow{t in tim};      # power available at time t
param P_pow{t in tim};                  #cost of power at time t
param C_sw{p in pump};               #Maintanence cost for pump switch
param n_pu{p in pump};#efficiency of pump
param dens;    #density of water
# Variables

var x{p in pump,t in tim} binary; #1 is pump is switched on, 0 otherwise
var h_node{t in tim} >= 0; #the head at the node
var Q{p in pump, t in tim} >= 0; #discharge from pump p at time t
var h_pump{t in tim}>=0;     #head of pump p at time t
var tank_level {t in tim} >=0;

#Objective
```

```
minimize Cost: sum{t in tim, p in pump} (((x[p,t] * Q[p,t] * h_pump[t] *
P_pow[t])/n_pu[p]) + (C_sw[p] * max (0, (x[p,t] - (if t>1 then x[p,t-1] else 0)))));
subject to
HeadNod {t in tim}: (Y_tank + ElevT - ElevN) - (2*f_eq*L_eq/(g*D_eq^3)*(sum{p in
pump} Q[p,t])^2 );
subject to
minlevel_tank {t in tim}: tank_level[t] >= Y_tank;
subject to
demand {t in tim} : Qd * pat[t] <= sum {p in pump} (Q[p,t] * x [p,t]);
subject to
level_tank {t in tim} : tank_level[t] = h_pump[t]-(ElevT + Z_pump + H_suc +
(2*f_suc*L_suc/(g*D_suc^3)*(sum{p in pump} Q[p,t])^2) +
(2*f_del*L_del/(g*D_del^3)*(sum{p in pump} Q[p,t])^2 ));
subject to
discharge_pump {t in tim, p in pump} : (Pow[t]/npump)*(3.6*0.746*(10^6))/(dens * g *
h_pump[t]);
subject to
headlim {t in tim}: hmin <=h_node[t] <= hmax;
```

APPENDIX 2

EPANET INPUT FILE FOR EXAMPLE WDS IN CHAPTER 4

```
[TITLE]


[JUNCTIONS]
;ID              Elev            Demand          Pattern
 I1              0.0000          0.000000        1               ;
 I7              0.0000          0.000000        1               ;
 I6              0.0000          0.000000        1               ;
 I3              0.0000          0.000000        1               ;
 I4              0.0000          0.000000        1               ;
 I5              0.0000          0.000000        1               ;
 I2              0.0000          0.000000        1               ;
 Pw1             0.0000          3211.545200     2               ;
 Pw2             0.0000          16334.617300    3               ;
 Pw3             0.0000          142.560330      4               ;
 Pw4             10.0000         208.976910      5               ;
 Pw5             10.0000         651.102170      6               ;
 City1           20.0000         0.000000        1               ;
 1.4             20.0000         4000.000000     1               ;
 1.3             20.0000         4000.000000     1               ;
 1.2             20.0000         4000.000000     1               ;
 1.1             20.0000         4000.000000     1               ;
 City2           10.0000         0.000000        1               ;
 2.5             20.0000         3000.000000     1               ;
 2.4             20.0000         3000.000000     1               ;
 2.1             20.0000         3000.000000     1               ;
 2.2             20.0000         3000.000000     1               ;
 2.3             20.0000         3000.000000     1               ;
 J1              0.0000          0.000000        1               ;
 J2              0.0000          0.000000        1               ;
 J3              0.0000          0.000000        1               ;
 J5              0.0000          0.000000        1               ;
 J4              0.0000          0.000000        1               ;
 2               20.0000         0.000000        1               ;
 4               10.0000         0.000000        1               ;

[RESERVOIRS]
;ID              Head            Pattern
 WaterSource             0.0000                          ;
 WWTP                    0.0000                          ;

[TANKS]
;ID              Elevation       InitLevel       MinLevel        MaxLevel        Diameter
        MinVol          VolCurve
 3               55.0000         40.0000         0.0000          70.0000         140
        0.0000                          ;
```

```
 1             55.0000      40.0000      0.0000       60.0000      120
       0.0000                      ;

[PIPES]
;ID            Node1        Node2        Length       Diameter     Roughness
       MinorLoss    Status
 ML1           I1           Pw1          5280.0000    18.0000      120.0000
       0.0000       CV      ;
 ML2           I2           Pw2          2640.0000    18.0000      120.0000
       0.0000       CV      ;
 ML3           I3           Pw3          10560.0000   18.0000      120.0000
       0.0000       CV      ;
 ML6           I6           Pw5          10560.0000   18.0000      120.0000
       0.0000       CV      ;
 ML7           I7           Pw4          15840.0000   18.0000      120.0000
       0.0000       CV      ;
 IL1.1         City1        1.1          7920.0000    36.0000      120.0000
       0.0000       Open    ;
 IL1.2         City1        1.2          7920.0000    36.0000      120.0000
       0.0000       Open    ;
 IL1.3         City1        1.3          7920.0000    36.0000      120.0000
       0.0000       Open    ;
 IL1.4         City1        1.4          7920.0000    36.0000      120.0000
       0.0000       Open    ;
 IL2.1         City2        2.1          5280.0000    36.0000      120.0000
       0.0000       Open    ;
 IL2.2         City2        2.2          5280.0000    36.0000      120.0000
       0.0000       Open    ;
 IL2.3         City2        2.3          5280.0000    36.0000      120.0000
       0.0000       Open    ;
 IL2.4         City2        2.4          5280.0000    36.0000      120.0000
       0.0000       Open    ;
 IL2.5         City2        2.5          5280.0000    36.0000      120.0000
       0.0000       Open    ;
 RW2           J2           Pw2          13200.0000   16.0000      120.0000
       0.0000       CV      ;
 RW1           J1           Pw1          10560.0000   16.0000      120.0000
       0.0000       CV      ;
 RW3           J3           Pw3          15840.0000   16.0000      120.0000
       0.0000       CV      ;
 RW5           J5           Pw5          21120.0000   16.0000      120.0000
       0.0000       CV      ;
 RW4           J4           Pw4          15840.0000   16.0000      120.0000
       0.0000       CV      ;
 SL1           2            City1        100.0000     50.0000      120.0000
       0.0000       CV      ;
```

```
 ML4            I4              3               7920.0000       60.0000         120.0000
        0.0000          CV      ;
 SL2            4               City2           100.0000        36.0000         120.0000
        0.0000          CV      ;
 ML5            I5              1               10560.0000      60.0000         100.0000
        0.0000          CV      ;
 1              3               2               100.0000        50.0000         120.0000
        0.0000          Open    ;
 2              1               4               100.0000        36.0000         120.0000
        0.0000          Open    ;

[PUMPS]
;ID             Node1           Node2           Parameters
 WP1.1          WaterSource             I1              HEAD 1          ;
 WP2.1          WaterSource             I2              HEAD 5          ;
 WP3.1          WaterSource             I3              HEAD 2          ;
 WP4.1          WaterSource             I4              HEAD 3          ;
 WP6.1          WaterSource             I6              HEAD 2          ;
 WP7.1          WaterSource             I7              HEAD 2          ;
 RWP1.1                 WWTP            J1              HEAD 1          ;
 RWP2.2                 WWTP            J2              HEAD 5          ;
 RWP3.1                 WWTP            J3              HEAD 2          ;
 RWP4.1                 WWTP            J5              HEAD 2          ;
 RWP5.1                 WWTP            J4              HEAD 2          ;
 WP4.2          WaterSource             I4              HEAD 3          ;
 WP5.1          WaterSource             I5              HEAD 1          ;
 WP5.2          WaterSource             I5              HEAD 1          ;
 WP4.3          WaterSource             I4              HEAD 3          ;
 WP5.3          WaterSource             I5              HEAD 3          ;
 WP1.2          WaterSource             I1              HEAD 1          ;
 WP2.3          WaterSource             I2              HEAD 5          ;
 WP3.2          WaterSource             I3              HEAD 2          ;
 WP7.2          WaterSource             I7              HEAD 2          ;
 WP6.2          WaterSource             I6              HEAD 2          ;
 RWP4.2                 WWTP            J5              HEAD 2          ;
 RWP5.2                 WWTP            J4              HEAD 2          ;
 RWP1.2                 WWTP            J1              HEAD 1          ;
 RWP2.3                 WWTP            J2              HEAD 5          ;
 RWP3.2                 WWTP            J3              HEAD 2          ;
 WP2.2          WaterSource             I2              HEAD 5          ;
 RWP2.1                 WWTP            J2              HEAD 5          ;

[VALVES]
;ID             Node1           Node2           Diameter        Type    Setting
        MinorLoss
```

```
[TAGS]

[DEMANDS]
;Junction        Demand          Pattern         Category

[STATUS]
;ID              Status/Setting

[PATTERNS]
;ID              Multipliers
;
 1               0.6233          0.1861          0.4234          0.3617          0.3349
        0.3234
 1               0.5281          0.4518          0.6122          0.8674          0.5885
        0.6630
 1               1.0412          0.6031          0.4500          0.7780          0.1524
        0.2699
 1               0.1372          0.2574          0.1392          0.7128          0.5208
        1.1517
;
;
2       0.18088         0.14670         0.14670         0.16379         0.18088
        0.21506
2       0.18088         0.18088         0.18088         0.18088         0.18088
        0.14670
2       0.16379         0.11253         0.09544         0.11253         0.12962
        0.14670
2       0.18088         0.18088         0.18088         0.18088         0.14670
        0.11253
2       0.11253
;
3       0.14252         0.11602         0.11602         0.12927         0.14252
        0.16902
3       0.14252         0.14252         0.14252         0.14252         0.14252
        0.11602
3       0.12927         0.08951         0.07626         0.08951         0.10276
        0.11602
3       0.14252         0.14252         0.14252         0.14252         0.11602
        0.08951
3       0.08951
;
4       0.28006         0.22605         0.22605         0.25305         0.28006
        0.33407
4       0.28006         0.28006         0.28006         0.28006         0.28006
        0.22605
```

```
4	0.25305	0.17204	0.14503	0.17204	0.19904
	0.22605
4	0.28006	0.28006	0.28006	0.28006	0.22605
	0.17204
4	0.17204
;
5	0.01000	0.01000	0.01000	0.01000	0.01000
	0.01000
5	0.01000	0.01000	0.01000	0.01000	0.01000
	0.01000
5	0.01000	0.01000	0.01000	0.01000	0.01000
	0.01000
5	0.01000	0.01000	0.01000	0.01000	0.01000
	0.01000
5	0.01000
;
6	0.16973	0.13778	0.13778	0.15376	0.16973
	0.20167
6	0.16973	0.16973	0.16973	0.16973	0.16973
	0.13778
6	0.15376	0.10584	0.08986	0.10584	0.12181
	0.13778
6	0.16973	0.16973	0.16973	0.16973	0.13778
	0.10584
6	0.10584
;
;2		0.6		0.8		0.8		0.9		1.0
	1.2
;2		1.0		1.0		1.0		1.0		1.0
	0.8
;2		0.9		0.6		0.5		0.6		0.7
	0.8
;2		1.0		1.0		1.0		1.0		0.8
	0.6
;
;patternStop

[CURVES]
;ID		X-Value		Y-Value
;PUMP: PUMP: PUMP:
 1		5500		75.0000
;PUMP: PUMP: PUMP:
 2		0.0000		150.0000
 2		600.0000	131.2500
 2		1050.0000	102.7500
;PUMP: PUMP: PUMP:
```

```
 3              8000            75
;PUMP: PUMP: PUMP:
 4              10000           180.0000
;PUMP: PUMP: PUMP:
 5              11000           150

[CONTROLS]
PUMP WP1.1 CLOSED AT TIME 0:00
PUMP WP2.1 CLOSED AT TIME 0:00
PUMP WP3.1 CLOSED AT TIME 0:00
PUMP WP4.1 CLOSED AT TIME 0:00
PUMP WP6.1 OPEN AT TIME 0:00
PUMP WP7.1 OPEN AT TIME 0:00
PUMP RWP1.1 OPEN AT TIME 0:00
PUMP RWP2.2 OPEN AT TIME 0:00
PUMP RWP3.1 OPEN AT TIME 0:00
PUMP RWP4.1 CLOSED AT TIME 0:00
PUMP RWP5.1 OPEN AT TIME 0:00
PUMP WP4.2 OPEN AT TIME 0:00
PUMP WP5.1 OPEN AT TIME 0:00
PUMP WP5.2 CLOSED AT TIME 0:00
PUMP WP4.3 OPEN AT TIME 0:00
PUMP WP5.3 CLOSED AT TIME 0:00
PUMP WP1.2 CLOSED AT TIME 0:00
PUMP WP2.3 CLOSED AT TIME 0:00
PUMP WP3.2 OPEN AT TIME 0:00
PUMP WP7.2 OPEN AT TIME 0:00
PUMP WP6.2 OPEN AT TIME 0:00
PUMP RWP4.2 CLOSED AT TIME 0:00
PUMP RWP5.2 OPEN AT TIME 0:00
PUMP RWP1.2 CLOSED AT TIME 0:00
PUMP RWP2.3 OPEN AT TIME 0:00
PUMP RWP3.2 OPEN AT TIME 0:00
PUMP WP2.2 OPEN AT TIME 0:00
PUMP RWP2.1 OPEN AT TIME 0:00
PUMP WP1.1 CLOSED AT TIME 1:00
PUMP WP2.1 CLOSED AT TIME 1:00
PUMP WP3.1 OPEN AT TIME 1:00
PUMP WP4.1 OPEN AT TIME 1:00
PUMP WP6.1 OPEN AT TIME 1:00
PUMP WP7.1 OPEN AT TIME 1:00
PUMP RWP1.1 CLOSED AT TIME 1:00
PUMP RWP2.2 CLOSED AT TIME 1:00
PUMP RWP3.1 OPEN AT TIME 1:00
PUMP RWP4.1 CLOSED AT TIME 1:00
PUMP RWP5.1 CLOSED AT TIME 1:00
```

PUMP WP4.2 OPEN AT TIME 1:00
PUMP WP5.1 OPEN AT TIME 1:00
PUMP WP5.2 OPEN AT TIME 1:00
PUMP WP4.3 CLOSED AT TIME 1:00
PUMP WP5.3 CLOSED AT TIME 1:00
PUMP WP1.2 CLOSED AT TIME 1:00
PUMP WP2.3 OPEN AT TIME 1:00
PUMP WP3.2 OPEN AT TIME 1:00
PUMP WP7.2 OPEN AT TIME 1:00
PUMP WP6.2 OPEN AT TIME 1:00
PUMP RWP4.2 CLOSED AT TIME 1:00
PUMP RWP5.2 OPEN AT TIME 1:00
PUMP RWP1.2 OPEN AT TIME 1:00
PUMP RWP2.3 OPEN AT TIME 1:00
PUMP RWP3.2 OPEN AT TIME 1:00
PUMP WP2.2 CLOSED AT TIME 1:00
PUMP RWP2.1 OPEN AT TIME 1:00
PUMP WP1.1 OPEN AT TIME 2:00
PUMP WP2.1 OPEN AT TIME 2:00
PUMP WP3.1 OPEN AT TIME 2:00
PUMP WP4.1 CLOSED AT TIME 2:00
PUMP WP6.1 OPEN AT TIME 2:00
PUMP WP7.1 CLOSED AT TIME 2:00
PUMP RWP1.1 OPEN AT TIME 2:00
PUMP RWP2.2 OPEN AT TIME 2:00
PUMP RWP3.1 OPEN AT TIME 2:00
PUMP RWP4.1 OPEN AT TIME 2:00
PUMP RWP5.1 OPEN AT TIME 2:00
PUMP WP4.2 CLOSED AT TIME 2:00
PUMP WP5.1 CLOSED AT TIME 2:00
PUMP WP5.2 CLOSED AT TIME 2:00
PUMP WP4.3 CLOSED AT TIME 2:00
PUMP WP5.3 CLOSED AT TIME 2:00
PUMP WP1.2 CLOSED AT TIME 2:00
PUMP WP2.3 OPEN AT TIME 2:00
PUMP WP3.2 OPEN AT TIME 2:00
PUMP WP7.2 OPEN AT TIME 2:00
PUMP WP6.2 OPEN AT TIME 2:00
PUMP RWP4.2 CLOSED AT TIME 2:00
PUMP RWP5.2 CLOSED AT TIME 2:00
PUMP RWP1.2 CLOSED AT TIME 2:00
PUMP RWP2.3 CLOSED AT TIME 2:00
PUMP RWP3.2 CLOSED AT TIME 2:00
PUMP WP2.2 CLOSED AT TIME 2:00
PUMP RWP2.1 OPEN AT TIME 2:00
PUMP WP1.1 OPEN AT TIME 3:00

PUMP WP2.1 OPEN AT TIME 3:00
PUMP WP3.1 OPEN AT TIME 3:00
PUMP WP4.1 OPEN AT TIME 3:00
PUMP WP6.1 OPEN AT TIME 3:00
PUMP WP7.1 OPEN AT TIME 3:00
PUMP RWP1.1 OPEN AT TIME 3:00
PUMP RWP2.2 CLOSED AT TIME 3:00
PUMP RWP3.1 CLOSED AT TIME 3:00
PUMP RWP4.1 CLOSED AT TIME 3:00
PUMP RWP5.1 CLOSED AT TIME 3:00
PUMP WP4.2 OPEN AT TIME 3:00
PUMP WP5.1 OPEN AT TIME 3:00
PUMP WP5.2 CLOSED AT TIME 3:00
PUMP WP4.3 CLOSED AT TIME 3:00
PUMP WP5.3 OPEN AT TIME 3:00
PUMP WP1.2 OPEN AT TIME 3:00
PUMP WP2.3 CLOSED AT TIME 3:00
PUMP WP3.2 OPEN AT TIME 3:00
PUMP WP7.2 OPEN AT TIME 3:00
PUMP WP6.2 OPEN AT TIME 3:00
PUMP RWP4.2 CLOSED AT TIME 3:00
PUMP RWP5.2 CLOSED AT TIME 3:00
PUMP RWP1.2 CLOSED AT TIME 3:00
PUMP RWP2.3 CLOSED AT TIME 3:00
PUMP RWP3.2 CLOSED AT TIME 3:00
PUMP WP2.2 OPEN AT TIME 3:00
PUMP RWP2.1 CLOSED AT TIME 3:00
PUMP WP1.1 OPEN AT TIME 4:00
PUMP WP2.1 CLOSED AT TIME 4:00
PUMP WP3.1 OPEN AT TIME 4:00
PUMP WP4.1 OPEN AT TIME 4:00
PUMP WP6.1 OPEN AT TIME 4:00
PUMP WP7.1 CLOSED AT TIME 4:00
PUMP RWP1.1 OPEN AT TIME 4:00
PUMP RWP2.2 OPEN AT TIME 4:00
PUMP RWP3.1 OPEN AT TIME 4:00
PUMP RWP4.1 CLOSED AT TIME 4:00
PUMP RWP5.1 OPEN AT TIME 4:00
PUMP WP4.2 CLOSED AT TIME 4:00
PUMP WP5.1 OPEN AT TIME 4:00
PUMP WP5.2 OPEN AT TIME 4:00
PUMP WP4.3 CLOSED AT TIME 4:00
PUMP WP5.3 OPEN AT TIME 4:00
PUMP WP1.2 CLOSED AT TIME 4:00
PUMP WP2.3 OPEN AT TIME 4:00
PUMP WP3.2 CLOSED AT TIME 4:00

PUMP WP7.2 CLOSED AT TIME 4:00
PUMP WP6.2 CLOSED AT TIME 4:00
PUMP RWP4.2 OPEN AT TIME 4:00
PUMP RWP5.2 OPEN AT TIME 4:00
PUMP RWP1.2 CLOSED AT TIME 4:00
PUMP RWP2.3 CLOSED AT TIME 4:00
PUMP RWP3.2 OPEN AT TIME 4:00
PUMP WP2.2 CLOSED AT TIME 4:00
PUMP RWP2.1 CLOSED AT TIME 4:00
PUMP WP1.1 CLOSED AT TIME 5:00
PUMP WP2.1 OPEN AT TIME 5:00
PUMP WP3.1 CLOSED AT TIME 5:00
PUMP WP4.1 OPEN AT TIME 5:00
PUMP WP6.1 OPEN AT TIME 5:00
PUMP WP7.1 OPEN AT TIME 5:00
PUMP RWP1.1 CLOSED AT TIME 5:00
PUMP RWP2.2 OPEN AT TIME 5:00
PUMP RWP3.1 OPEN AT TIME 5:00
PUMP RWP4.1 OPEN AT TIME 5:00
PUMP RWP5.1 OPEN AT TIME 5:00
PUMP WP4.2 OPEN AT TIME 5:00
PUMP WP5.1 CLOSED AT TIME 5:00
PUMP WP5.2 CLOSED AT TIME 5:00
PUMP WP4.3 OPEN AT TIME 5:00
PUMP WP5.3 OPEN AT TIME 5:00
PUMP WP1.2 OPEN AT TIME 5:00
PUMP WP2.3 OPEN AT TIME 5:00
PUMP WP3.2 OPEN AT TIME 5:00
PUMP WP7.2 OPEN AT TIME 5:00
PUMP WP6.2 OPEN AT TIME 5:00
PUMP RWP4.2 OPEN AT TIME 5:00
PUMP RWP5.2 OPEN AT TIME 5:00
PUMP RWP1.2 OPEN AT TIME 5:00
PUMP RWP2.3 CLOSED AT TIME 5:00
PUMP RWP3.2 OPEN AT TIME 5:00
PUMP WP2.2 OPEN AT TIME 5:00
PUMP RWP2.1 CLOSED AT TIME 5:00
PUMP WP1.1 CLOSED AT TIME 6:00
PUMP WP2.1 OPEN AT TIME 6:00
PUMP WP3.1 OPEN AT TIME 6:00
PUMP WP4.1 CLOSED AT TIME 6:00
PUMP WP6.1 CLOSED AT TIME 6:00
PUMP WP7.1 OPEN AT TIME 6:00
PUMP RWP1.1 CLOSED AT TIME 6:00
PUMP RWP2.2 OPEN AT TIME 6:00
PUMP RWP3.1 CLOSED AT TIME 6:00

PUMP RWP4.1 OPEN AT TIME 6:00
PUMP RWP5.1 OPEN AT TIME 6:00
PUMP WP4.2 OPEN AT TIME 6:00
PUMP WP5.1 OPEN AT TIME 6:00
PUMP WP5.2 OPEN AT TIME 6:00
PUMP WP4.3 CLOSED AT TIME 6:00
PUMP WP5.3 CLOSED AT TIME 6:00
PUMP WP1.2 CLOSED AT TIME 6:00
PUMP WP2.3 OPEN AT TIME 6:00
PUMP WP3.2 CLOSED AT TIME 6:00
PUMP WP7.2 CLOSED AT TIME 6:00
PUMP WP6.2 OPEN AT TIME 6:00
PUMP RWP4.2 OPEN AT TIME 6:00
PUMP RWP5.2 CLOSED AT TIME 6:00
PUMP RWP1.2 OPEN AT TIME 6:00
PUMP RWP2.3 OPEN AT TIME 6:00
PUMP RWP3.2 OPEN AT TIME 6:00
PUMP WP2.2 OPEN AT TIME 6:00
PUMP RWP2.1 OPEN AT TIME 6:00
PUMP WP1.1 OPEN AT TIME 7:00
PUMP WP2.1 OPEN AT TIME 7:00
PUMP WP3.1 OPEN AT TIME 7:00
PUMP WP4.1 OPEN AT TIME 7:00
PUMP WP6.1 OPEN AT TIME 7:00
PUMP WP7.1 OPEN AT TIME 7:00
PUMP RWP1.1 OPEN AT TIME 7:00
PUMP RWP2.2 OPEN AT TIME 7:00
PUMP RWP3.1 CLOSED AT TIME 7:00
PUMP RWP4.1 CLOSED AT TIME 7:00
PUMP RWP5.1 OPEN AT TIME 7:00
PUMP WP4.2 CLOSED AT TIME 7:00
PUMP WP5.1 CLOSED AT TIME 7:00
PUMP WP5.2 OPEN AT TIME 7:00
PUMP WP4.3 OPEN AT TIME 7:00
PUMP WP5.3 OPEN AT TIME 7:00
PUMP WP1.2 CLOSED AT TIME 7:00
PUMP WP2.3 OPEN AT TIME 7:00
PUMP WP3.2 OPEN AT TIME 7:00
PUMP WP7.2 CLOSED AT TIME 7:00
PUMP WP6.2 OPEN AT TIME 7:00
PUMP RWP4.2 OPEN AT TIME 7:00
PUMP RWP5.2 OPEN AT TIME 7:00
PUMP RWP1.2 OPEN AT TIME 7:00
PUMP RWP2.3 OPEN AT TIME 7:00
PUMP RWP3.2 OPEN AT TIME 7:00
PUMP WP2.2 CLOSED AT TIME 7:00

PUMP RWP2.1 CLOSED AT TIME 7:00
PUMP WP1.1 OPEN AT TIME 8:00
PUMP WP2.1 CLOSED AT TIME 8:00
PUMP WP3.1 OPEN AT TIME 8:00
PUMP WP4.1 OPEN AT TIME 8:00
PUMP WP6.1 OPEN AT TIME 8:00
PUMP WP7.1 OPEN AT TIME 8:00
PUMP RWP1.1 OPEN AT TIME 8:00
PUMP RWP2.2 OPEN AT TIME 8:00
PUMP RWP3.1 OPEN AT TIME 8:00
PUMP RWP4.1 CLOSED AT TIME 8:00
PUMP RWP5.1 OPEN AT TIME 8:00
PUMP WP4.2 CLOSED AT TIME 8:00
PUMP WP5.1 OPEN AT TIME 8:00
PUMP WP5.2 OPEN AT TIME 8:00
PUMP WP4.3 OPEN AT TIME 8:00
PUMP WP5.3 OPEN AT TIME 8:00
PUMP WP1.2 CLOSED AT TIME 8:00
PUMP WP2.3 OPEN AT TIME 8:00
PUMP WP3.2 CLOSED AT TIME 8:00
PUMP WP7.2 OPEN AT TIME 8:00
PUMP WP6.2 CLOSED AT TIME 8:00
PUMP RWP4.2 CLOSED AT TIME 8:00
PUMP RWP5.2 CLOSED AT TIME 8:00
PUMP RWP1.2 OPEN AT TIME 8:00
PUMP RWP2.3 CLOSED AT TIME 8:00
PUMP RWP3.2 CLOSED AT TIME 8:00
PUMP WP2.2 OPEN AT TIME 8:00
PUMP RWP2.1 CLOSED AT TIME 8:00
PUMP WP1.1 OPEN AT TIME 9:00
PUMP WP2.1 CLOSED AT TIME 9:00
PUMP WP3.1 CLOSED AT TIME 9:00
PUMP WP4.1 OPEN AT TIME 9:00
PUMP WP6.1 CLOSED AT TIME 9:00
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[RULES]

[ENERGY]
 Global Efficiency	75.0000
 Global Price	0
 Demand Charge	0.0000

[EMITTERS]
;Junction	Coefficient

[QUALITY]
;Node	InitQual

[SOURCES]
;Node	Type	Quality	Pattern
 WaterSource	CONCEN	200	1
 WWTP	CONCEN	3000	1

```
[REACTIONS]
;Type           Pipe/Tank       Coefficient


[REACTIONS]
 Order Bulk             1.00
 Order Tank             1.00
 Order Wall             1
 Global Bulk            0.000000
 Global Wall            0.000000
 Limiting Potential     0
 Roughness Correlation          0

[MIXING]
;Tank           Model

[TIMES]
 Duration       25:00
 Hydraulic Timestep     1:00
 Quality Timestep       0:05
 Pattern Timestep       1:00
 Pattern Start  0:00
 Report Timestep        1:00
 Report Start   0:00
 Start ClockTime        8:00 AM
 Statistic      NONE

[REPORT]
 Status         YES
 Summary                YES
 LINKS          ALL
 NODES          ALL
 Page           0
 ENERGY                 Yes

[OPTIONS]
 Units          GPM
 Headloss               H-W
 Specific Gravity       1.000000
 Viscosity              1.000000
 Trials         40
 Accuracy               0.00100000
 CHECKFREQ              2
 MAXCHECK               10
 DAMPLIMIT              0.00000000
```

Unbalanced	Continue 10
Pattern	1
Demand Multiplier	1.0000
Emitter Exponent	0.5000
Quality	Chemical mg/L
Diffusivity	8
Tolerance	0.01000000

APPENDIX 3

EPANET INPUT FILE FOR EXAMPLE WDS IN CHAPTER 5

[TITLE]
North Marin Water District Zone I

[JUNCTIONS]

;ID	Elev	Demand	Pattern	
10	147	0.00		;
15	32	620.00	3	;
20	129	0.00		;
35	12.5	1856	4	;
40	131.9	0		;
50	116.5	0		;
60	0	0		;
61	0	0		;
101	42	189.95	1	;
103	43	133.2	1	;
105	28.5	135.37	1	;
107	22	54.64	1	;
109	20.3	231.4	1	;
111	10	141.94	1	;
113	2	20.01	1	;
115	14	52.1	1	;
117	13.6	117.71	1	;
119	2	176.13	1	;
120	0	0		;
121	-2	41.63		;
123	11	1859	2	;
125	11	45.6	1	;
127	56	17.66	1	;
129	51	0		;
131	6	42.75	1	;
139	31	5.89	1	;
141	4	9.85	1	;
143	-4.5	6.2	1	;
145	1	27.63	1	;
147	18.5	8.55	1	;
149	16	27.07	1	;
151	33.5	144.48	1	;
153	66.2	44.17	1	;
157	13.1	51.79	1	;
159	6	41.32	1	;
161	4	15.8	1	;
163	5	9.42	1	;
164	5	0		;
166	-2	2.6	1	;
167	-5	14.56	1	;
169	-5	0		;

171	-4	39.34	1		;
173	-4	0			;
177	8	58.17	1		;
179	8	0			;
181	8	0			;
183	11	0			;
184	16	0			;
185	16	25.65	1		;
187	12.5	10	6		;
189	4	107.92	1		;
191	25	81.9	1		;
193	18	71.31	1		;
195	15.5	0			;
197	23	17.04	1		;
199	-2	119.32	1		;
201	0.1	44.61	1		;
203	2	4643	5		;
204	21	0			;
205	21	65.36	1		;
206	1	0			;
207	9	69.39	1		;
208	16	0			;
209	-2	0.87	1		;
211	7	8.67	1		;
213	7	13.94	1		;
215	7	92.19	1		;
217	6	24.22	1		;
219	4	4.32	1		;
225	8	22.8	1		;
229	10.5	64.18	1		;
231	5	16.48	1		;
237	14	15.61	1		;
239	13	44.61	1		;
241	13	0			;
243	14	4.34	1		;
247	18	70.38	1		;
249	18	10		7	;
251	30	24.16	1		;
253	36	54.52	1		;
255	27	40.39	1		;
257	17	0			;
259	25	10		8	;
261	0	0			;
263	0	0			;
265	0	10		9	;
267	21	0			;

```
 269            0             0                            ;
 271            6             0                            ;
 273            8             0                            ;
 275            10            10             10            ;
 P1             0             1000         6               ;
 P2             0             1000         7               ;
 P3             0             1000         8               ;
 P4             0             1000         9               ;
 P5             0             1000         10              ;

[RESERVOIRS]
;ID             Head          Pattern
 4              220.0                      ;
 5              167.0                      ;

[TANKS]
;ID             Elevation     InitLevel    MinLevel     MaxLevel     Diameter
        MinVol        VolCurve
 1              131.9         13.1         0.1          32.1         85
        0.1                          ;
 2              116.5         23.5         6.5          40.3         50
        0.1                          ;
 3              129.0         29.0         4.0          35.5         164
        0.1                          ;

[PIPES]
;ID             Node1         Node2        Length       Diameter     Roughness
        MinorLoss     Status
 20             3             20           99           24           199
        0             Open    ;
 40             1             40           99           24           199
        0             Open    ;
 50             2             50           99           24           199
        0             Open    ;
 60             4             60           1231         24           140
        0             Open    ;
 101            10            101          14200        18           110
        0             Open    ;
 103            101           103          1350         16           130
        0             Open    ;
 105            101           105          2540         12           130
        0             Open    ;
 107            105           107          1470         12           130
        0             Open    ;
 109            103           109          3940         16           130
        0             Open    ;
```

111	109	111	2000	12	130	0	Open	;
112	115	111	1160	12	130	0	Open	;
113	111	113	1680	12	130	0	Open	;
114	115	113	2000	8	130	0	Open	;
115	107	115	1950	8	130	0	Open	;
116	113	193	1660	12	130	0	Open	;
117	263	105	2725	12	130	0	Open	;
119	115	117	2180	12	130	0	Open	;
120	119	120	730	12	130	0	Open	;
121	120	117	1870	12	130	0	Open	;
122	121	120	2050	8	130	0	Open	;
123	121	119	2000	30	141	0	Open	;
125	123	121	1500	30	141	0	Open	;
129	121	125	930	24	130	0	Open	;
131	125	127	3240	24	130	0	Open	;
133	20	127	785	20	130	0	Open	;
135	127	129	900	24	130	0	Open	;
137	129	131	6480	16	130	0	Open	;
145	129	139	2750	8	130	0	Open	;
147	139	141	2050	8	130	0	Open	;
149	143	141	1400	8	130	0	Open	;
151	15	143	1650	8	130	0	Open	;
153	145	141	3510	12	130	0	Open	;

155	147	145	2200	12	130	0	Open	;
159	147	149	880	12	130	0	Open	;
161	149	151	1020	8	130	0	Open	;
163	151	153	1170	12	130	0	Open	;
169	125	153	4560	8	130	0	Open	;
171	119	151	3460	12	130	0	Open	;
173	119	157	2080	30	141	0	Open	;
175	157	159	2910	30	141	0	Open	;
177	159	161	2000	30	141	0	Open	;
179	161	163	430	30	141	0	Open	;
180	163	164	150	14	130	0	Open	;
181	164	166	490	14	130	0	Open	;
183	265	169	590	30	141	0	Open	;
185	167	169	60	8	130	0	Open	;
186	187	204	99.9	8	130	0	Open	;
187	169	171	1270	30	141	0	Open	;
189	171	173	50	30	141	0	Open	;
191	271	171	760	24	130	0	Open	;
193	35	181	30	24	130	0	Open	;
195	181	177	30	12	130	0	Open	;
197	177	179	30	12	130	0	Open	;
199	179	183	210	12	130	0	Open	;
201	40	179	1190	12	130	0	Open	;

202	185	184	99.9	8	130	0	Open	;
203	183	185	510	8	130	0	Open	;
204	184	205	4530	12	130	0	Open	;
205	204	185	1325	12	130	0	Open	;
207	189	183	1350	12	130	0	Open	;
209	189	187	500	8	130	0	Open	;
211	169	269	646	12	130	0	Open	;
213	191	187	2560	12	130	0	Open	;
215	267	189	1230	12	130	0	Open	;
217	191	193	520	12	130	0	Open	;
219	193	195	360	12	130	0	Open	;
221	161	195	2300	8	130	0	Open	;
223	197	191	1150	12	130	0	Open	;
225	111	197	2790	12	130	0	Open	;
229	173	199	4000	24	141	0	Open	;
231	199	201	630	24	141	0	Open	;
233	201	203	120	24	130	0	Open	;
235	199	273	725	12	130	0	Open	;
237	205	207	1200	12	130	0	Open	;
238	207	206	450	12	130	0	Open	;
239	275	207	1430	12	130	0	Open	;
240	206	208	510	12	130	0	Open	;
241	208	209	885	12	130	0	Open	;

243	209	211	1210	16	130	0	Open	;
245	211	213	990	16	130	0	Open	;
247	213	215	4285	16	130	0	Open	;
249	215	217	1660	16	130	0	Open	;
251	217	219	2050	14	130	0	Open	;
257	217	225	1560	12	130	0	Open	;
261	213	229	2200	8	130	0	Open	;
263	229	231	1960	12	130	0	Open	;
269	211	237	2080	12	130	0	Open	;
271	237	229	790	8	130	0	Open	;
273	237	239	510	12	130	0	Open	;
275	239	241	35	12	130	0	Open	;
277	241	243	2200	12	130	0	Open	;
281	241	247	445	12	130	0	Open	;
283	239	249	430	12	130	0	Open	;
285	247	249	10	12	130	0	Open	;
287	247	255	1390	10	130	0	Open	;
289	50	255	925	10	130	0	Open	;
291	255	253	1100	10	130	0	Open	;
293	255	251	1100	8	130	0	Open	;
295	249	251	1450	12	130	0	Open	;
297	120	257	645	8	130	0	Open	;
299	257	259	350	8	130	0	Open	;

301	259	263	1400	8	130	0	Open	;
303	257	261	1400	8	130	0	Open	;
305	117	261	645	12	130	0	Open	;
307	261	263	350	12	130	0	Open	;
309	265	267	1580	8	130	0	Open	;
311	193	267	1170	12	130	0	Open	;
313	269	189	646	12	130	0	Open	;
315	181	271	260	24	130	0	Open	;
317	273	275	2230	8	130	0	Open	;
319	273	205	645	12	130	0	Open	;
321	163	265	1200	30	141	0	Open	;
323	201	275	300	12	130	0	Open	;
325	269	271	1290	8	130	0	Open	;
329	61	123	45500	30	140	0	Open	;
p1	187	P1	10	12	130	0	Open	;
p3	259	P3	10	12	130	0	Open	;
p4	265	P4	10	12	130	0	Open	;
p5	275	P5	10	12	130	0	Open	;
p2	249	P2	10	12	130	0	Open	;

[PUMPS]

;ID	Node1	Node2	Parameters		
10	5	10	HEAD 10	SPEED 1	;
335	60	61	HEAD 335	SPEED 1	;

[VALVES]

;ID Node1 Node2 Diameter Type Setting
MinorLoss

[TAGS]
NODE P1 P1

[DEMANDS]
;Junction Demand Pattern Category

[STATUS]
;ID Status/Setting
10 Closed
335 Closed

[PATTERNS]
;ID Multipliers
;
1 1.34 1.94 1.46 1.44 0.76 0.92
1 0.85 1.07 0.96 1.10 1.08 1.19
1 1.16 1.08 0.96 0.83 0.79 0.74
1 0.64 0.64 0.85 0.96 1.24 1.67

2 0 0 0 0 0 0.656
2 0 0 0 1.0037 0.988 0.978
2 0.978 0.98 0.980 0.977 0.981 0.976
2 0.986 0.984 0.984 0.975 0.989 1.00

3 1 1 1 1 1 0.580
3 0.580 0 0 0 0 0.580
3 0.580 0.580 0.580 0.580 0 0
3 0 0 0 0 0.580 0.580

4 0.882 0.919 0.926 0.926 0.964 0.980
4 0.957 0.992 0.977 0.983 1.00 0.970
4 0.980 0.934 0.896 0.872 0.869 0.872
4 0.870 0.887 0.876 0.876 0.900 0.898

5 0.956 0.975 0.971 0.986 0.975 0.986
5 0.984 0.993 1.00 1.00 0.989 0.993
5 0.975 0.973 0.958 0.956 0.958 0.960
5 0.956 0.951 0.940 0.947 0.962 0.964
6
7
8
9
10

```
;2		0.6		0.8		0.8		0.9		1.0
	1.2
;2		1.0		1.0		1.0		1.0		1.0
	0.8
;2		0.9		0.6		0.5		0.6		0.7
	0.8
;2		1.0		1.0		1.0		1.0		0.8
	0.6
;
;patternStop

[CURVES]
;ID		X-Value		Y-Value
;PUMP: PUMP: PUMP: PUMP:
 10		0		104
 10		3000		92
 10		6000		63
;PUMP: PUMP: PUMP: PUMP:
 335		0		200
 335		8000		138
 335		14000		86

[CONTROLS]
pump 10 open at time 0
pump 335 open at time 0
pump 10 open at time 1
pump 335 open at time 1
pump 10 open at time 2
pump 335 open at time 2
pump 10 open at time 3
pump 335 open at time 3
pump 10 open at time 4
pump 335 open at time 4
pump 10 open at time 5
pump 335 open at time 5
pump 10 open at time 6
pump 335 open at time 6
pump 10 open at time 7
pump 335 open at time 7
pump 10 open at time 8
pump 335 open at time 8
pump 10 open at time 9
pump 335 open at time 9
pump 10 open at time 10
pump 335 open at time 10
pump 10 open at time 11
```

pump 335 open at time 11
pump 10 open at time 12
pump 335 open at time 12
pump 10 open at time 13
pump 335 open at time 13
pump 10 open at time 14
pump 335 open at time 14
pump 10 open at time 15
pump 335 open at time 15
pump 10 open at time 16
pump 335 open at time 16
pump 10 open at time 17
pump 335 open at time 17
pump 10 open at time 18
pump 335 open at time 18
pump 10 open at time 19
pump 335 open at time 19
pump 10 open at time 20
pump 335 open at time 20
pump 10 open at time 21
pump 335 open at time 21
pump 10 open at time 22
pump 335 open at time 22
pump 10 open at time 23
pump 335 open at time 23
pump 10 open at time 24
pump 335 open at time 24
pump 10 open at time 25
pump 335 open at time 25
pump 10 open at time 26
pump 335 open at time 26
pump 10 open at time 27
pump 335 open at time 27
pump 10 open at time 28
pump 335 open at time 28
pump 10 open at time 29
pump 335 open at time 29
pump 10 open at time 30
pump 335 open at time 30
pump 10 open at time 31
pump 335 open at time 31
pump 10 open at time 32
pump 335 open at time 32
pump 10 open at time 33
pump 335 open at time 33
pump 10 open at time 34

```
pump 335 open at time 34
pump 10 open at time 35
pump 335 open at time 35
pump 10 open at time 36
pump 335 open at time 36
pump 10 open at time 37
pump 335 open at time 37
pump 10 open at time 38
pump 335 open at time 38
pump 10 open at time 39
pump 335 open at time 39
pump 10 open at time 40
pump 335 open at time 40
pump 10 open at time 41
pump 335 open at time 41
pump 10 open at time 42
pump 335 open at time 42
pump 10 open at time 43
pump 335 open at time 43
pump 10 open at time 44
pump 335 open at time 44
pump 10 open at time 45
pump 335 open at time 45
pump 10 open at time 46
pump 335 open at time 46
pump 10 open at time 47
pump 335 open at time 47
pump 10 open at time 48
pump 335 open at time 48
;link 335 OPEN IF Node 1 BELOW 17.1
;Link 335 CLOSED IF Node 1 ABOVE 19.1
;Link 10 CLOSED IF Node 1 BELOW 17.1
;Link 10 OPEN IF Node 1 ABOVE 19.1


[RULES]
RULE 1
IF TANK 1 LEVEL ABOVE 19.1
THEN PUMP 335 STATUS IS CLOSED
AND LINK 10 STATUS IS OPEN

RULE 2
IF TANK 1 LEVEL BELOW 17.1
THEN PUMP 335 STATUS IS OPEN
AND LINK 10 STATUS IS CLOSED
```

```
[ENERGY]
 Global Efficiency      75
 Global Price           0
 Demand Charge          0

[EMITTERS]
;Junction        Coefficient

[QUALITY]
;Node            InitQual

[SOURCES]
;Node            Type            Quality          Pattern

[REACTIONS]
;Type            Pipe/Tank       Coefficient


[REACTIONS]
 Order Bulk             1
 Order Tank             1
 Order Wall             1
 Global Bulk            0
 Global Wall            0
 Limiting Potential     0
 Roughness Correlation          0

[MIXING]
;Tank            Model

[TIMES]
 Duration               24:00
 Hydraulic Timestep     1:00
 Quality Timestep       0:05
 Pattern Timestep       1:00
 Pattern Start          0:00
 Report Timestep        1:00
 Report Start           0:00
 Start ClockTime        8:00 AM
 Statistic        NONE

[REPORT]
 Status           YES
 Summary                YES
```

LINKS ALL
NODES ALL
Page 0
ENERGY Yes

[OPTIONS]
Units GPM
Headloss H-W
Specific Gravity 0.998
Viscosity 1
Trials 40
Accuracy 0.001
CHECKFREQ 2
MAXCHECK 10
DAMPLIMIT 0
Unbalanced Continue 10
Pattern 1
Demand Multiplier 1.0
Emitter Exponent 0.5
Quality None mg/L
Diffusivity 1
Tolerance 0.01

APPENDIX 4

MATLAB FILES FOR OPTIMIZATION-SIMULATION MODEL IN CHAPTER 6

A4.1. Main Execution File

```
global d; global ncont;global iter; global y_func;
iter = 1;
y_func = [];
%inpFileName = 'SanJuan_Mays_15';
%inpFileName = 'SanJuan_Paper25%_248';
%inpFileName = 'SanJuan_Paper40%';
inpFileName = 'SanJuan_SOA';
%   Load EPANET toolkit
inpName = strcat('..\EPANET\',inpFileName, '.inp');

version='epanet2';                  % Version of EPANET used
d = epanet(inpName);       % create an instance of EPANET in MATLABN
[controls, controlMatrix n] = getControlMatrix(d);
ncont = 10;
initPopulation = controlMatrix(ncont+1:(ncont)*24+ncont,4);
initPopulation = initPopulation';
lb = [];
lb(1:n) = 0;
ub = [];
ub(1:n) = 1;
nvars = ncont*24;
popSize = 30;
genCount = 1000;
[y] =  SanJuanOptions(nvars,lb,ub,d,initPopulation,popSize,genCount);

controlMatrix(ncont+1:n,4) = [y y y y y y y y y y y y y y] ;
setControlMatrix(d, n, controlMatrix);
d.saveInputFile('output.inp');
```

A4.2. Genetic Algorithm Options Function File

```
function [X1] = SanJuanOptions(nvars,lb,ub,d,initPopulation,popSize,genCount)
%% This is an auto generated MATLAB file from Optimization Tool.
%% Start with the default options
options = gaoptimset;
%% Modify options setting
options = gaoptimset(options,'PopulationSize', popSize);
options = gaoptimset(options,'EliteCount',6);
% options = gaoptimset(options,'EliteCount',6);
options = gaoptimset(options,'Generations', genCount);
options = gaoptimset(options,'CrossoverFraction',0.6);
%options = gaoptimset(options,'TimeLimit', 60);
options = gaoptimset(options,'FitnessLimit', 1*1e-10);
```

```
options = gaoptimset(options,'TolFun', 1e-20);
options = gaoptimset(options,'TolCon', 1e-100);
options = gaoptimset(options,'StallGenLimit',30);
options = gaoptimset(options,'InitialPopulation',initPopulation);
% options = gaoptimset(options,'PopInitRange',[0;4]);
options = gaoptimset(options,'Display', 'iter');
% options = gaoptimset(options,'UseParallel',true);
%options = gaoptimset(options,'PlotFcns', {  @gaplotbestf @gaplotbestindiv
@gaplotgenealogy @gaplotrange @gaplotscorediversity });
 [X1] = ga(@fitnessfunction2,nvars,[],[],[],[],lb,ub,[],[1:nvars],options);
End
```

A4.3. Genetic Algorithm Fitness Function File

```
function [y] = fitnessfunction2(N)
global d; global ncont;global iter; global y_func;

[controls, controlMatrix n] = getControlMatrix(d);

controlMatrix(ncont+1:n,4) = [N N N N N N N N N N N N N N] ;

setControlMatrix(d, n, controlMatrix);

[P, T, D, H, F] = hydroSolve(d);

[a nodes] = size(P);
PressureConstraints = zeros(a, nodes);
i = 0;
strT = 0;
for i = 5:nodes - 16
   for t = 1:a
   if T(t,1) == 3*86400
      strT = t;
   end
   PressureConstraints(t,i) =  max(0, 30 - P(t,i)) + 10^-2*max(0, P(t,i) - 80) + max(0,
P(t,i) - 100);
   end
end
for i = 1:nodes
   for t = 1:a
   if T(t,1) == 3*86400
      strT = t;
   end
   PressureConstraints(t,i) = 1000000* max(0, 0 - P(t,i)) + PressureConstraints(t,i);
   if PressureConstraints(t,i) < 1
      PressureConstraints(t,i) = 0;
```

```
    end
    end
end
OutletValves = [1782 1787 1762];
OutletValves1 = [1783 1785 1789];
InletValves = [971, 1786, 1763];
InletValves1 = [1726, 1784, 1788];
%OutletValves = [1799 1795 1790];
%OutletValves1 = [1799 1795 1790];
%InletValves = [1798, 1794, 1791];
%InletValves1 = [1798, 1794, 1791];
%   Determine hourly time pattern
[timePatternLoc] = getTimePatterns(T);
[a_t b_t] = size(timePatternLoc);
outFlows = [];
outQ = [];
inQ = [];
inFlows = [];
for i = 1:b_t
    for j = 1:3
        outFlows(i,j) = F(timePatternLoc(i),OutletValves(j))*60;
        outQ(i,j) = F(timePatternLoc(i),OutletValves1(j));
        inFlows(i,j) = F(timePatternLoc(i), InletValves(j))*60;
        inQ(i,j) = F(timePatternLoc(i), InletValves1(j));
    end
end
for i = 1:b_t
       H_red(i,:) = H(timePatternLoc(i),:);
       P_red(i,:) = P(timePatternLoc(i),:);
end
totOutflows = [];
totOutflows(1) = sum(outFlows(73:96,1));
totOutflows(2) = sum(outFlows(73:96,2));
totOutflows(3) = sum(outFlows(73:96,3));
% h3 = max(H(strT:end,1434)) - 214.02;
% h2 = max(H(strT:end,1433)) - 214;
% %h1 = max(H(strT:end,1431)) - 205.58;
% h1 = max(H(strT:end,1432)) - 222.58;
% TotalTankVolumes = [pi/4*(68.3)^2*h1 pi/4*(40)^2*h2 pi/4*(67.8455)^2*h3]*7.49;
% turnover = 0.25;
% turnoverlim = 0.50;
% tankVolumeConssup = turnoverlim*TotalTankVolumes - totOutflows;
% tankVolumeConss = totOutflows - turnover*TotalTankVolumes;
% tankVolumeConsup = sum(abs(min(0, tankVolumeConssup)));
% tankVolumeCons = sum(abs(min(0, tankVolumeConss)));
```

```
% %Hconstraint = max(0,250.3 -max(H(strT:end,1432)))+abs(min(H(strT:end,1431)) -
236.5)+abs(min(H(strT:end,1430)) - 226.68)+abs(min(H(strT:end,1432)) - 242.72);
% Hconstraint = max(0,251 -max(H(strT:end,1432))) + max(0,244 -
max(H(strT:end,1433))) + max(0,251 -
max(H(strT:end,1434)));%+abs(min(H(strT:end,1431)) -
229)+abs(min(H(strT:end,1430)) - 219.79);%+abs(min(H(strT:end,1432)) - 235.52);
pumpHours = sum(sum(N));
[pow_pump] = pumpPower(T,H,F);
% y = sum(sum(N))+0.001*sum(sum(pow_pump))+ 500*10^15*Hconstraint +
100000000000000*(sum(sum(PressureConstraints)))+10^10*(0.1*tankVolumeCons+0.1
*tankVolumeConsup);
% % y_func(iter,1) = iter;
% y_func(iter,2) = y;
% [a_y b_y] = size(y_func);
% for i = 1:a_y
%     for j = 1:b_y
%     end
% end
%
% dlmwrite('fitnessfunc.txt',y_func);
%y =  Hconstraint + 100000000000000*(sum(sum(PressureConstraints)))
pmax = [];pmin = [];
P_a = [P_red(:,1:943) P_red(:,945:1420)];
for i = 1:b_t
  pmax(i) = max(P_a(i,:));
  pmin(i) = min(P_a(i,:));
end
M = reshape(N,ncont,24);
%  Solve for quality
[Qual, T_qual] = qualitySolve(d);
[timePatternLoc_qual] = getTimePatterns(T_qual);
Qual_red = [];
[a_q b_q] = size(timePatternLoc_qual);
for i = 1:b_q
   Qual_red(i,:) = Qual(timePatternLoc_qual(i), :);
end
[P_resilience, Q_resilience] = resilience(d, P_red, Qual_red, pow_pump, H_red);
iter = iter+1;
end
```

APPENDIX 5

MATLAB CODE FOR INFRASTRUCTURAL-OPERATIONAL RESILIENCE COMPUTATIONS

A5.1. Main Resilience Computations Function

```
Res = [];
Res_top = []
timePeriods = 672;
startIndex = 1;
startDir = pwd;
inpFileName = 'TestSystem1';
global Mconn_n; global Mbet_n; global a_pump;
   cd('..\MATLABWS');
   [d] = loadEpanet(inpFileName, num2str(1));
   cd(startDir);
NodeNames = d.NodeNameID;
[b, nNodes]= size(NodeNames);
Mconn = ones(1, nNodes);

NodeLinkConn = d.NodesConnectingLinksIndex;
[bNodes, b] = size(NodeLinkConn);
linkDiams = d.LinkDiameter;
a_size = linkDiams/max(linkDiams);
pumpIndex = d.LinkPumpIndex;
a_type = zeros(1,bNodes);
[p_a, p_b] = size(pumpIndex);
for i = 1:p_b
   a_type(pumpIndex(i)) = 1;
end
a_link = a_size + a_type;
for i = 1:bNodes
   Mconn(NodeLinkConn(i,1)) = Mconn(NodeLinkConn(i,1)) +1;
end
Mbet = ones(1, nNodes);
for i = 1:bNodes
   Mbet(NodeLinkConn(i,1)) = Mbet(NodeLinkConn(i,1)) + a_link(NodeLinkConn(i,2));
end
Mbet_n = Mbet/max(Mbet);
maxMconn = max(Mconn);
Mconn_n = Mconn/maxMconn;
i = 0;
pump_size = [10500 1050 20000 10000 11000];
curve_index = d.HeadCurveIndex;
for i = 1:p_b
   a_pump(i) = pump_size(curve_index(i));
end
a_pump = (a_pump/sum(a_pump))*28;
for i = 1:timePeriods
   timeP = num2str(i)
```

```
    cd('..\MATLABWS');
    [d] = loadEpanet(inpFileName, num2str(i));
    cd(startDir);
    pumpLoadFile = strcat('allPumpLoads_',num2str(i),'.txt');
    pressFile = strcat('pressures_',num2str(i),'.txt');
    p = load(pressFile);
    pump_pow = load(pumpLoadFile);
    [res_P(i) res_Pw(i) res_D(i) Res_top(i)] = resilience_top(p, pump_pow,
startIndex,timeP);
    showtop = Res_top(i)
    fileName = strcat('..\EPANET\resilience_top', '_', num2str(i), '.txt');
    fid = fopen(fileName, 'wt');
    fprintf(fid, '%.2f\t', Res_top(i));
    fclose(fid);
    [resilience_P(i) resilience_Pw(i) resilience_D(i) Res(i)] = resilience_aydin(p,
pump_pow, startIndex,timeP);
    showRes = Res(i)
    fileName = strcat('..\EPANET\resilience_aydin', '_', num2str(i), '.txt');
    fid = fopen(fileName, 'wt');
    fprintf(fid, '%.2f\t', Res(i));
    fclose(fid);
    if res_P(i) < resilience_P(i)
        showtop = Res_top(i)
    end
end

% for i = 1:timePeriods
%     timeP = num2str(i)
%     cd('..\MATLABWS');
%     [d] = loadEpanet(inpFileName, num2str(i));
%     cd(startDir);
%     pumpLoadFile = strcat('allPumpLoads_',num2str(i),'.txt');
%     pressFile = strcat('pressures_',num2str(i),'.txt');
%     p = load(pressFile);
%     pump_pow = load(pumpLoadFile);
%     [resilience_P(i) resilience_Pw(i) resilience_D(i) Res(i)] = resilience_aydin(p,
pump_pow, startIndex,timeP)
%     fileName = strcat('..\EPANET\resilience_aydin', '_', num2str(i), '.txt');
%     fid = fopen(fileName, 'wt');
%     fprintf(fid, '%.2f\t', Res(i));
%     fclose(fid);
% end

    time = [1:timePeriods];
resilience_aydin = Res;
resilience_top = Res_top;
```

```
s = figure;
figure(s)
plot(time,resilience_aydin(1:timePeriods),'--ob');
ylim([0 1]);
xl = xlabel('Time (hrs)');
set(xl,'FontSize',16);
yl = ylabel('Resilience');
set(yl,'FontSize',16);
pName = 'Resilience_aydin';
saveas(s,strcat(pName,'.png'))
close('all');
s = figure;
figure(s)
plot(time,resilience_top(1:timePeriods),'--ob');
ylim([0 1]);
xl = xlabel('Time (hrs)');
set(xl,'FontSize',16);
yl = ylabel('Resilience');
set(yl,'FontSize',16);
pName = 'Resilience_top';
saveas(s,strcat(pName,'.png'))
close('all');
s = figure;
figure(s)
plot(time,res_Pw(1:timePeriods),'--ob');
ylim([0 1]);
xl = xlabel('Time (hrs)');
set(xl,'FontSize',16);
yl = ylabel('Resilience');
set(yl,'FontSize',16);
pName = 'Resilience_top_Pw';
saveas(s,strcat(pName,'.png'))
close('all');
    s = figure;
figure(s)
plot(time,res_D(1:timePeriods),'--ob');
ylim([0 1]);
xl = xlabel('Time (hrs)');
set(xl,'FontSize',16);
yl = ylabel('Resilience');
set(yl,'FontSize',16);
pName = 'Resilience_top_D';
saveas(s,strcat(pName,'.png'))
close('all');
s = figure;
figure(s)
```

```
plot(time,res_P(1:timePeriods),'--ob');
xl = xlabel('Time (hrs)');
set(xl,'FontSize',16);
yl = ylabel('Resilience');
set(yl,'FontSize',16);
pName = 'Resilience_top_P';
saveas(s,strcat(pName,'.png'))
close('all');
s = figure;
figure(s)
plot(time,resilience_Pw(1:timePeriods),'--ob');
ylim([0 1]);
xl = xlabel('Time (hrs)');
set(xl,'FontSize',16);
yl = ylabel('Resilience');
set(yl,'FontSize',16);
pName = 'Resilience_aydin_pw';
saveas(s,strcat(pName,'.png'))
close('all');
 s = figure;
figure(s)
plot(time,resilience_D(1:timePeriods),'--ob');
ylim([0 1]);
xl = xlabel('Time (hrs)');
set(xl,'FontSize',16);
yl = ylabel('Resilience');
set(yl,'FontSize',16);
pName = 'Resilience_aydin_D';
saveas(s,strcat(pName,'.png'))
close('all');
  s = figure;
figure(s)
plot(time,resilience_P(1:timePeriods),'--ob');
xl = xlabel('Time (hrs)');
set(xl,'FontSize',16);
yl = ylabel('Resilience');
set(yl,'FontSize',16);
pName = 'Resilience_aydin_P';
saveas(s,strcat(pName,'.png'))
close('all');
```

A5.2 Operational Resilience Computation Function

```
function [resilience_P resilience_Pw resilience_D R] = resilience_aydin(P, pow_pump, startIndex,tp)
```

```
global pwMax;global tanknodes;
tanknodes = [1 2 3 4 5 6 7 8 9 10 11 12 19 20 21 22 23 24 25 26 27 28 29 30 31 32 33 34
35 36 37 38 39];

startDir = pwd;
cd('..\PSLF');
pumpPowerFile = strcat('pumpPowers_', tp, '.txt');
pwMax = load(pumpPowerFile);
pwMax = 1000*pwMax';
cd(startDir);
  %   Compute Pressure penalty voilations
[pressX pressY] = size(P);
pressVoi = 0;
[n m] = size(tanknodes);
pressVoi = [];
k = 0;
unsatis_Press = zeros(1,39);
for pressi = 1:pressX - startIndex
    k = k+1;
    for pressj = 1:pressY
        flag = 1;
        for i = 1:m
            if pressj == tanknodes(i)
                pressVoi(pressi,pressj) = 1;
                flag = 0;
            end
        end
        if flag == 1
            if P(pressi+startIndex,pressj) < 19
                pressVoi(pressi,pressj) = 0;
                unsatis_Press(1,pressj) = unsatis_Press(1,pressj) + 1;
%            elseif P(pressi+startIndex,pressj) <40
%                pressVoi(pressi, pressj) = (P(pressi, pressj) - 20)/20;
            elseif (P(pressi+startIndex, pressj)>19)&&(P(pressi, pressj)<100)
                pressVoi(pressi, pressj) = 1;
%            elseif (P(pressi+startIndex,pressj) > 80)&&(P(pressi,pressj) <100)
%                pressVoi(pressi, pressj) = (100-(P(pressi,pressj)))/20;
            elseif P(pressi+startIndex,pressj) > 100
                pressVoi(pressi, pressj) = 0;
                unsatis_Press(1,pressj) = unsatis_Press(1,pressj) + 1;
            end
        end
    end
end

[rel_i rel_j] = size(pressVoi);
```

```
relP_den = rel_i * rel_j;
unsatis_P = relP_den - sum(sum(pressVoi));

relP = sum(sum(pressVoi))/relP_den;
for i = 1:rel_i-1
   for j = 1:rel_j
   satis_P(i,j) = pressVoi(i+1,j)-pressVoi(i,j);
   if satis_P(i,j) < 0
      satis_P(i,j) = 0;
   end
   end
end
M_den = 0;

   for j = 1:rel_j
      satisnode_P(j) = sum(satis_P(:,j));
      resnode_P(j) = satisnode_P(j)/unsatis_Press(1,j);
      if unsatis_Press(1,j) ==0
         resnode_P(j) = 1;
      end
      if unsatis_Press(1,j) ==1
         resnode_P(j) = 0;
      end
   end
resilience_P = sum(resnode_P)/pressY;
%   Compute Pressure power voilations
powFunc = [];
[powX powY] = size(pow_pump);

for powi = 1:powX
   for powj = 1:powY
      powFunc(powi,powj) = 1;
      if (pow_pump(powi,powj) > pwMax(powi,powj))
         powFunc(powi,powj) = 0;
      end
   end
end

[relPw_i relPw_j] = size(powFunc);
relPw_den = relPw_i*relPw_j;
unsatis_Pw = relPw_den - sum(sum(powFunc));

relPw = sum(sum(powFunc))/relPw_den;
origpattern = load(strcat('origPattern_',tp,'.txt'));
X3 = load(strcat('newNodeDemandMultipliers_',tp,'.txt'));
   corr = [];
```

```
    [i_pat, j_pat] = size(X3);
    for i = 1:6
        for j = 1:j_pat
            if X3(i,j) == origpattern(i,j)
                corr(i,j) = 1;
            elseif X3(i,j) < origpattern(i,j)
                corr(i,j) = X3(i,j)/origpattern(i,j);
            end
        end
    end
demandSatisfaction = corr;

[relD_i relD_j] = size(demandSatisfaction);
relD_den = relD_i*relD_j;
relD = sum(sum(demandSatisfaction))/relD_den;
unsatis_D = relD_den - sum(sum(demandSatisfaction));
prod = [];

%   Total Resilience

reliability = (relP + relPw + relD)/3;

for i = 1:relPw_i-1
    for j = 1:relPw_j
    satis_Pw(i,j) = powFunc(i+1,j)-powFunc(i,j);
    if satis_Pw(i,j) < 0
        satis_Pw(i,j) = 0;
    end
    end
end

resilience_Pw = sum(sum(satis_Pw))/unsatis_Pw;
if unsatis_Pw == 0
    resilience_Pw = 1;
end

for i = 1:relD_i
    satis_D(i) = demandSatisfaction(i,1);
end

resilience_D = sum(satis_D)/relD_i;

R = (resilience_P + resilience_Pw + resilience_D)/3;

End
```