THESIS

MODELING ARTIFICIAL GROUNDWATER RECHARGE AND LOW-HEAD HYDROELECTRIC PRODUCTION: A CASE STUDY OF SOUTHERN PAKISTAN

Submitted by

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ABSTRACT

MODELING ARTIFICIAL GROUNDWATER RECHARGE AND LOW-HEAD HYDROELECTRIC PRODUCTION: A CASE STUDY OF SOUTHERN PAKISTAN

DHA City Karachi (DCK), a city designed for approximately one million people, is envisaged to become a satellite city to the second largest city in the world, Karachi, which has a population of 25 million. The upcoming city is located 21 miles north of main metropolitan Karachi in the arid southern part of Pakistan. The region receives little rainfall with an annual average of 217mm and temperatures ranging from an average of 88°F in the summers to 68°F in the winters. The town has a projected water demand in the fully developed stage of 45 Million Gallons per Day (MGD) and 500 Mega Watts (MW) of electricity. Since water and electricity are prized and expensive commodities in the region, alternate and renewable sources of both need to be explored for DCK to meet its goal of sustainability and conservation. Two options for these sources, artificial recharge and hydroelectric production, are explored in this study.

Artificial recharge to replenish groundwater resources is becoming more common in arid areas. In this thesis, the capacity of small lakes to produce significant seepage and recharge to the underlying aquifer within city limits is explored for DCK. The lakes are fed by treated effluent from Sewage Treatment Plants (STP), that ponds through the vadose zone to the water table. Artificial recharge and resulting groundwater flow within the aquifer is simulated using a three-dimensional groundwater flow model (MODFLOW). A variety of pumping scenarios are explored to determine the quantity of groundwater that can be pumped for water supply. An optimal placement of 50 pumps throughout the city also is determined, with drawdown used as the variable

to be minimized so as to minimize pumping costs. In the fully developed stage of artificial recharge, the lakes feed almost 7.9 MGD of water to the aquifer, out of which 6.6 MGD can be pumped out and consumed sustainably on a daily basis through the 50 planned wells. Since DCK is to be developed and inhabited in 3 phases, analysis revealed that quantities of 1.4 and 3.5 MGD can be pumped out sustainably for the short and mid-term developmental plans.

A sustainable hydroelectric system was also designed for using the hydraulic structures of the small lakes. System control was introduced by application of Artificial Neural Networks (ANN) and Model Predictive Control (MPC) to maintain the hydroelectric potential and constant head against variation in flow as delivered from the STPs. The results show an output of 13.92 MW of green and sustainable hydroelectricity which can be produced at a very low cost.

A cost-benefit analysis projects a savings of $11,550 and $60,000 per day due to the artificial groundwater recharge and hydroelectric production respectively, with the cost of construction of these projects being paid off within 5 and 2 years at this rate, including the cost of operation and maintenance. Results, however, should be used with caution due to the preliminary nature of the models and calculations.

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DEDICATION

Firstly, I dedicate my MS Thesis to my parents and their fervor for not letting any stone unturned in the path of me getting the best available education in the world. I know that no dedications and appreciations can cater for the sacrifices they have made to make me what I am today. I am thankful to my siblings for being there during the thick and thin. I dedicate this to my paternal and maternal families, hoping that it will do the Siddiqui and Nomani family proud.

Last but not the least, I dedicate this work to the brave citizens of Pakistan who stand resilient in the face of oppression, cruelty and misfortune; always dreaming of a better tomorrow.

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CHAPTER 1.INTRODUCTION AND BACKGROUND INFORMATION

1.1 Geographical location

DHA City Karachi (DCK) is being erected as a satellite city to the second largest city in the world, Karachi. The city is envisioned to lessen the burden on the metropolitan area that has a population of 25 million, with a target population of 1 million to be developed in two portions: DCK North and DCK South, to be developed on 11,600 and 8,000 acres respectively. DCK South, with a target population of 600,000, is being developed first and is the center of focus for this study due to availability of data. The project is located 21 miles north of Karachi metropolitan on Highway M9 (see Figure 1). Karachi itself is located in the southern arid region of Pakistan on the coast of the Arabian Sea in South Asia. Due to the prevailing water and energy crises in the region, DCK needs to explore alternate and renewable sources to meet its goal of sustainability and conservation. Two sources are explored in this study: artificial recharge to the underlying aquifer, and hydroelectric generation. Both sources are developed using a network of small dams that capture treated effluent from Sewage Treatment Plants (STPs).

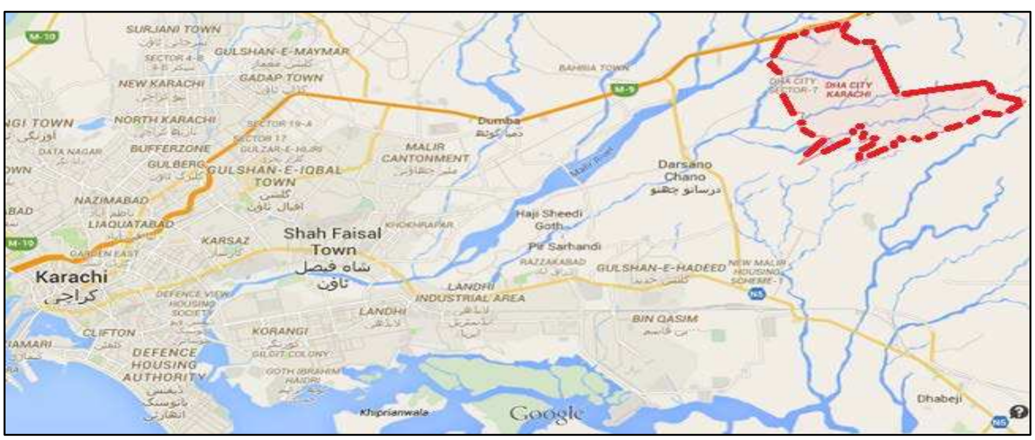

Figure 1: Area Map depicting geographical location of DCK with respect to Karachi.

1.2 Topography and Geology

DCK is located between what are locally called the *Nari* and *Gaj* formations towards the north of Karachi. The terrain is hilly with slopes ranging between 9 and 15% with elevation differences within the DCK reaching 120 meters. The *Nari* formation is composed of sandstone with shale whereas the *Gaj* formation is composed of sandstone with shale and minor limestone in the upper potion and massive limestone in the lower portion, some geologists have reported conglomerate at greater depths. Alluvial deposits can be seen on the beds of the non-perennial streams within DCK as it has been a floodplain over the years.

Hydrologically, it is located in the Upper Malir Basin and is home to the two small ephemeral streams which are locally known as *Abdar* and *Sukkun* whereas the two larger intermittent streams known as *Khadeji* and *Mol* are located to the south and north of DCK respectively, both of them drain out to the *Malir* river which eventually drains out to the Arabian sea. The depth of the water table within the aquifer system can range within the area between 3 to 30 meters.An average depth to water table of 21m was calculated from the borehole-log data for DCK. The main source of recharge is torrential rainfall and the main recharge area is located in the north-northwestern hills known as the *Kirthar* range. Karachi Water Supply Board gazette (2005) reported that they had inventoried around 350 wells in the region and which on average produce about 77 gallons per minute of groundwater.

There are no surface water resources close to DCK and it will primarily depend on buying water from the K3 and K4 schemes that currently supply water to Karachi. Due to the exposed nature of the two geological formations coupled with an arid climate, there is no thick vegetation/forest in the area. However, a few scattered shrubs can be found. Double-ring in filtrometer tests conducted at 5 sites in DCK revealed infiltration rates between 7 and 20 millimeters per hour. Data from the tests and the results are attached as Appendix A.

1.3 Climate

DCK is located in the arid southern region of Pakistan and is practically part of the Sindh desert. Because of being located on the coast of the Arabian Sea, it is also affected by the sea breeze. The humidity level in the city ranges between 64% and 90% throughout the year. The region receives little rainfall, with an annual average of 217mm and temperatures soaring to an average 88°F in the summers and 68°F during the winters (See Appendix A for meteorological data). Most of the rainfall is recorded in the *Monsoon season* which usually occurs during the months of July, August and September.

1.4 Design Philosophy and Master Plan of DCK

The entire planning approach for DCK is focused on Sustainable Design Principles, some of which include the concepts of connectivity, efficiency, renewability, minimization of externalities, and carrying capacity. These goals of sustainable design have been adopted in the overall planning approach by maintaining the ecological integrity of the site streams/ravines, reserved spaces and natural contours. They have been preserved and incorporated as prominent natural features in the master plan, by developing them as green fingers that also act as ventilation corridors for prevailing winds. There is a strong focus on the promotion of pedestrian movement along shaded and tree-lined pathways to improve the quality of life for DCK employees, visitors and residents.

DCK Master Plan consists of eight major land use types, which cumulatively covers approximately11,600 acres of land (see Table 1). From planning and development perspective, DCK is divided into three major regions viz. i) DCK Sectors (75%), ii) City Gateway and Down

City (6%), and iii)South Zone (11%). These zones are mainly divided by the planned major arterial roads and natural southern ridge. The DCK sectors are further sub-divided into sub-sectors on ekistics planning model (see Figures attached in Appendix C for detailed planning). Ekistics is an urban planning approach that considers geology, geography, environment, culture, customs and politics among other things. It has five basic elements: nature, Anthropos, society, shells, and networks.

Table 1: DCK Land Use Distribution

S. No	Land use Type	Area	
		Acres	%
1	Residential	4,354	37.53
2	Commercial	233	2.01
3	Mixed use(Res. cum Com.)	121	1.04
4	Amenities	543	4.68
5	Recreation	880	7.59
6	Utilities	376	3.24
7	Transportation	4,004	34.52
8	Agriculture & Reserved Spaces	1,090	9.40
Total		11,600	100.00

The ekistics unit size and design seek to create communities of walkable distances. By introducing multiple city centers with mixed – use live-work communities, the requirement of commuter's trips is reduced. Residential units are located within a radius of average half a mile from the civic center, and facilities of lower community classes are approached in a few minutes walking. A variety of facilities and uses is concentrated within each ekistics unit. This way all the main facilities are within walking distance from the residents. Figure 2 describes the master plan of DCK with respect to land use giving a wider picture of the design philosophy behind DCK.

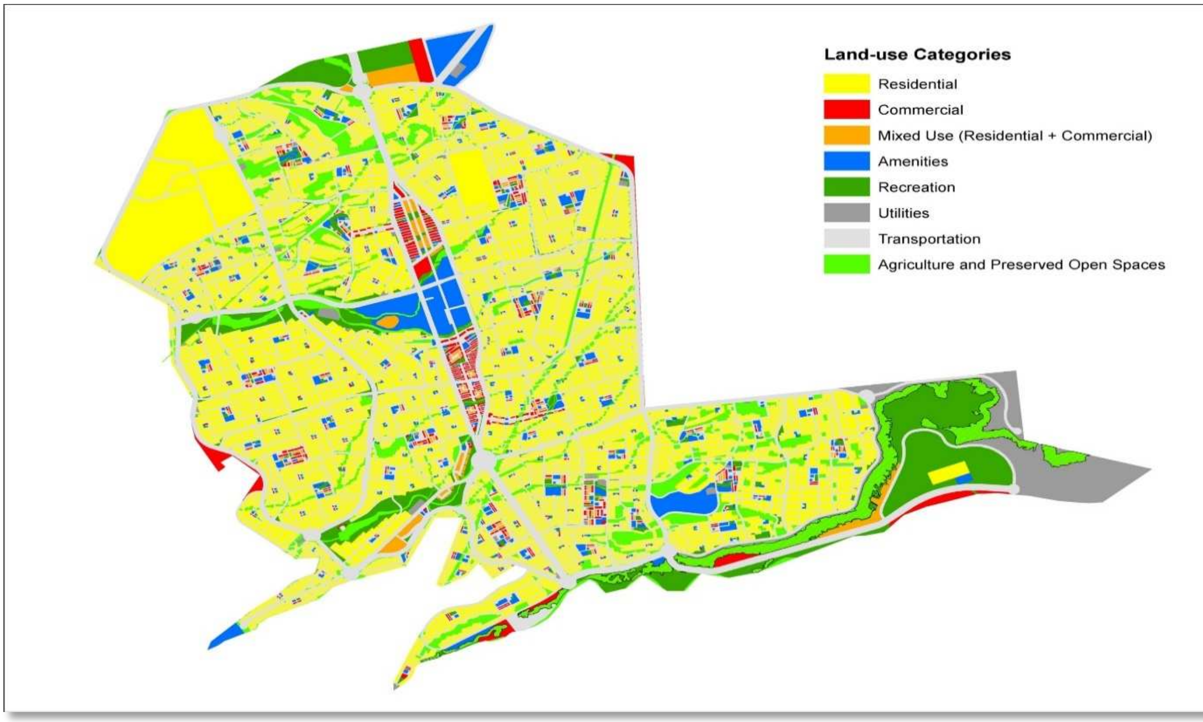

Figure 2: Area Map depicting geographical location of DCK with respect to Karachi

1.5 Scope of the Study

As an expensive and scarce commodity in the region, water is the main focus of this study. Groundwater storage within the underlying aquifer of the city area through artificial recharge has been suggested as an alternate source of water supply for DCK due to the lower probability of contamination and the lower rates of evaporative loss as compared to surface water storage. Spandre (2009) defined groundwater as the most significant alternate source of freshwater, lesser contaminated as compared to surface water, hence requiring less treatment processes. 30% of the worlds' freshwater is stored as groundwater. USGS, 1999 reported that in the US the quantity of water is 20 to 30 times more than in the surface water bodies. Artificial recharge of groundwater has become a common practice during the last few decades. Todd (1959) described artificial recharge as a dependable method for recovery of overexploited aquifers, storage of flood waters, and reduction of risk for land subsidence. Groundwater has been recharged directly to deeper aquifers artificially using injection wells, which however can be costly. Many studies have reported the use of treated municipal waste water for artificial recharge, which can of course be a costly expedition due to the cost of construction of such injection wells coupled with the cost of reverse pumping. Often, groundwater flow models are used to assess the performance of the system under current and predictive conditions of artificial recharge (Mohammadi, 2008), MODFLOW (Harbaugh, 2005) being one of the most often used. For DCK, a less costly system needs to be analyzed which recharges water naturally under the influence of head without any injection/pumping. A safe volume of water that can be extracted without stressing the aquifers also needs to be determined. In this thesis, the use of a network of small lakes throughout the DCK area is proposed as means to artificially recharge the underlying groundwater system.

Though the study primarily focuses on recharging groundwater by housing treated water at artificial lakes, it was proposed that the hydraulic structures at each lake can be utilized to produce green and sustainable energy through hydroelectric power production. With the increased demand of electricity, conventional sources such as oil, natural gas, and coal overload the environment with carbon emissions and other greenhouse gases causing danger to human health and environment (EPA, 2012). Hydroelectricity is a technology involving water dynamics and physics that has been used for over a century. Presently, 81% of the United States renewable energy is generated through hydropower that makes 10% of the country's total electricity (NHA, 2015). Small hydropower or micro hydel power projects are also slowly gaining popularity all over the world. As the proposed structures at the artificial lakes will receive water from the treatment plants, a system control needs to be designed using ANN (Artificial Neural Networks) and Model Predictive Control (MPC) to maintain the head (for artificial groundwater recharge) while also maintaining constant hydroelectric production.

1.6 Objectives

This study aims to achieve the following objectives to find a dependable, sustainable, and economic solution to DCK's water and energy problems:

- Estimate potential volume of groundwater recharge under the influence of the proposed network of artificial lakes within the DCK area. This has been done in Chapter 3. This is accomplished using a 3D MODFLOW model of the aquifer underlying the city's spatial area;
- Estimate the amount of water that can be sustainably extracted from the aquifer during the three planned developmental phases of DCK, it is also discussed in Chapter 3. This also will be accomplished using the MODFLOW model, with a set of production wells placed throughout the city;
- Find the location of the proposed production wells that minimizes groundwater head drawdown and thereby minimizes pumping costs, the finding of which can be found at the end of Chapter 3;
- Calculate the hydroelectric production potential of the small dams (see finding in Chapter 2) using ANN (Artificial Neural Networks) and Model Predictive Control (MPC) system control tools for the most optimized results; and
- Conduct cost-benefit analysis for the project as a decision support system for the stakeholders of DCK. The summary of results are discussed in Chapter 4.

Here is a flow chart to explain the working of the system:

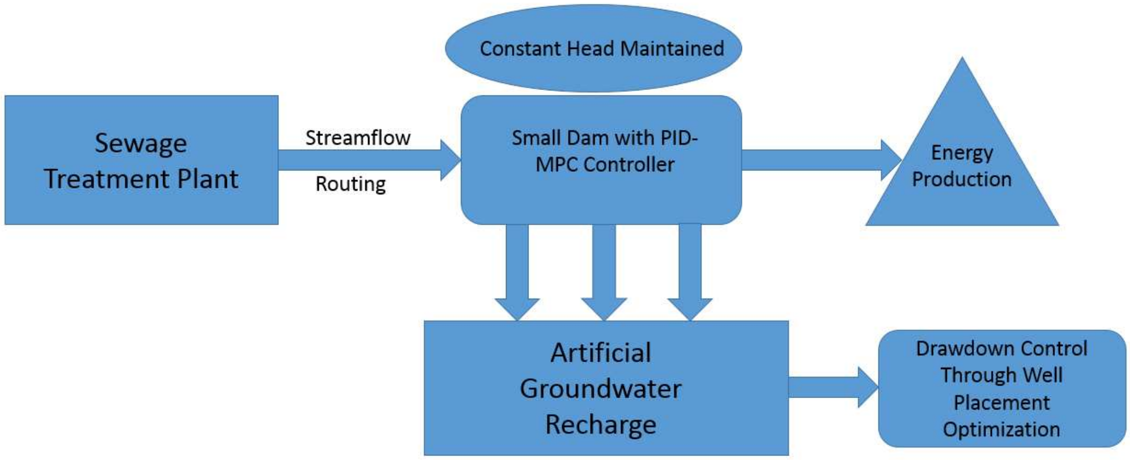

Figure 3: Schematic Diagram for the project.

CHAPTER 2.HYDROPOWER GENERATION

2.1 Introduction; Renewable energy

With the increased demand of electricity during recent decades, conventional sources such as oil, natural gas, and coal overload the environment with carbon emissions and other greenhouse gases causing danger to human health and environment (EPA, 2012). Therefore, renewable energy is a technology involving varies resources (water, wind, Sun, etc.) beneficial in terms of provision, economy, health, and environment. Hydroelectric, geothermal, wind energy, solar energy, and biomass facilitate are efficient remedies to control negative influence of anthropogenic activities on the globe. The factors of benefits of renewable energy are mentioned in the following paragraphs.

Many countries use coal power plants and/or gas power plants that account for global warming emissions. For the United States, 25% of emissions are from coal power and 6% are from gas power plants (EPA, 2012). In contrast, renewable energy produce little or no global warming emissions including manufacturing, operation and maintenance, and dismantling (IPCC, 2011). Numerically, coal emits 1.4-3.6 pounds of carbon dioxide equivalent per Kilowatt-hour (CO_2E/KWh) and gas emits approximately 1–2 pounds. Table 2 shows the approximation of emissions in carbon dioxide equivalent per kilo-watt hour (CO_2E/KWh) for renewable energy sources that are less as compared to non-renewables.

Table 2: Emission of carbon of various renewable energy sources

Renewable energy source	Carbon emission
Wind energy	0.02 – 0.04 CO_2E/KWh

Solar energy	0.07 – 0.2 CO_2E/KWh
Hydroelectricity	0.1 – 0.5 CO_2E/KWh
Geothermal	0.1 – 0.2 CO_2E/KWh

A study done by the U.S. Department of Energy's National Renewable Energy Laboratory demonstrated that using 80% of renewable energy for electricity by the year 2025 can reduce 81% of global warming emissions from electricity generation (NREL, 2012). Lehner et al. (2005) modelled the possible effects of global change on hydropower potential for Europe. They analyzed hydropower potential using future scenarios for water use and climate. On performing gross and realistic scenarios for a water GAP model, their results showed changes in discharge regime in future and unstable trends at regional level with 25% reduction in hydropower potential for South and South-eastern European countries like Greece, Italy, Turkey, Romania, Serbia, Macedonia and Romania.

Another factor involves public health, where air and water pollution caused by coal and natural gas effect neurological and respiratory systems. They are different from solar and wind energy sources that do not emit air pollution or operate on water causing stress on river pollution and related uses (drinking, agriculture). Rizk (2013) mentions that switching to renewable energy has reduced premature mortality rate and overall healthcare cost in the US.

In 2009, The Union of Concerned Scientists analyzed that by 2025 renewable energy industry will provide three times more jobs for generating the equal amount of electricity from fossil fuels (UCS, 2009). In Turkey, there is potential for renewable energy including water, solar, and geothermal. Out of 433 GWh/year of potential hydropower only 125 GWh/year can be generated economically and 36% of usable potential is already considered and power plants are under construction (Yuksel, 2010). In addition, this technique offers diversity in sources of electricity and stability in the prices for future. There are many successful projects being run across varies countries including Pakistan.

Mirza et al. (2007) discuss the need and importance of renewable energy sources in Pakistan for remote areas and increasing demand. They have defined hydropower, solar, and wind as prime and cost-effective sources focusing on wind power that has potential for current and future circumstances. Another study done by Muneer and Asif (2007) defines renewables as the most promising sources for Pakistan in terms of economy and sustainability. They compared wind and solar energy sources and found solar energy to be more economical and accessible than wind. Sahir and Qureshi (2008) and Chaudhry et al. (2009) have also investigated energy resources where they mentioned the prospective and barriers in implementation. This study is an intensive research focused on hydropower. The purpose of this chapter is to explain how the treated municipal waste water is being reclaimed and with the use of modern system control technology namely, Machine Predictive Control (MPC) and Artificial Neural Networks (ANN) provides for efficient way of optimizing maximum hydroelectric production in the known variability of incoming flow to the small dams throughout the day while maintaining the head at a constant level as required by our hypothesis for artificial groundwater recharge (discussed in chapter 3).

2.2 Small Hydropower Generation

Hydroelectricity is a technology involving water dynamics and physics that is being used for over a century. Presently, 81% of the United States renewable energy is generated through hydropower that makes 10% of the country’s total electricity (NHA, 2015). However, with the ongoing distribution and management critics, small-scale hydropower technique has been upfront as an environmentally sustainable solution. Generally, dams/reservoirs having height below 15 meters are classified as small scale hydropower plants. Apparently, there is no specific definition in terms of capacities. In Canada the upper limit capacity is defined between 20-25 MW where in US it is 30MW (IPCC, 2011). The National Renewable Energy Laboratory defines small hydropower systems as those to generate electricity from 0.01 MW to 30 MW (NREL, 2001). Zhou et al. (2009) provided analysis on existing small hydropower system in China and modeled for the future cost and capacity. They put forward the recommendation on enhancing and encouraging small hydropower generation.

Small hydropower is subdivided based on the capacities where mini hydro defines electricity <500KW and micro hydro is used to projects of <100KW. This emission-free technology generate electric supply for not only highly industrialized areas in China or United States but remote communities as well such as small lands or towns in Nepal, India and Peru. Huang and Yan (2009) provides information on China's hydropower potential. They elucidate that China is the richest in hydropower having theoretical potential of 694 GW with only 145.26 GW installed currently. Furthermore, they mentioned about the on-going projects and small hydropower plants to be the source of electricity for rural communities in China in future. Kong et al. (2015) provide information regarding on-going small hydropower projects in China and 412 counties that realized rural electrification by 2010.

Kosnik (2008) discussed hydropower as one of the mitigation strategies for United States to diminish the emissions from fossil fuels. He analyzed the potential by type (including upgrading large scale plants and developing small scale) and by state (28 states of U.S) and provided the ability to satisfy the renewable portfolio standard legislation, accordingly. His study strongly suggested viable sites for small hydropower. Subsequently, Kosnik (2010) determined the cost-effectiveness of the discovered sites in U.S. The cost-benefit perspective filtered some sites that provide economical and environment-friendly electric supply. In interest of the previous case, another study done by Kumar and Katoch (2015) highlighted the political, environmental, and social situation of the construction of larger dams and power plants in the western Himalayas. Due to the conflict, they provided Small Hydropower Plants as the solution to balance the storage of glacial water and ecology of flora and fauna.

A study by Kusre et al. (2010) provides an insight of hydropower potential at Kopili River basin in Assam, India. They used GIS and the hydrological model SWAT (Neitsch, 2005) for hydrologic assessment. From the results, 107 potential sites on 9 streams were identified throughout the watershed for generation of <0.5 MW of electricity. They recommended hydroelectricity as a resource to fulfill limited fossil fuels and environmental sustainability.

In Turkey, various hydro projects have been installed since a renewable energy law passed in 2005. Kucukali (2010) investigated the use of municipal water supply dams for hydropower generation. He showed that 45 municipal dams in Turkey can generate 173 GWh/year electric energy. He performed the analysis on Zonguldak Ulutan Dam as a case study. A study by Bakis and Demirbas (2010) shows that by the end of 2002, 70 projects have been operational from various dams and 203 more projects of <10MW are considered to be developed in the near future. In Nigeria, the growth of hydropower was 360% between 1971 and 2005; however only 5% of small hydropower potential has been tapped since 1964. The analysis done by Ohunakin et al. (2011) shows adequate operating and maintenance cost as compared to European countries. Also, the authors suggest involvement of government agencies for financial and tariff support.

2.3 Control Schemes Modeling

There are various factors concerning hydropower development, for example, environmental assessment needs to evaluate process-driven factors such as effect of flow variation on flora and fauna and fluctuations in natural water level. In order to attain a balance between environment and operational processes, models are developed that simulate and monitor the mentioned issues using control approach. Model Predictive Control (MPC) and Artificial Neural Network (ANN) are included in many control scheme models for hydropower development.

Model predictive control (MPC) is a modern automatic control technique which has found wide application is numerous engineering fields. It uses traditional feedback control techniques in several aspects. Partial Integral Differential (PID) control is very straight-forward but not robust, Linear Quadratic Regulator (LQR) control is optimal but for an infinite horizon, intelligent control requires prior training of the controller or at least persistent excitation which is also required for adaptive control. However, a disadvantage common to all of the controllers above is that there is no explicit mechanism to handle system or control constraints, and those which are optimal in some sense are optimized for an infinite future time. Real systems have constraints on the system states and controlled inputs as well, e.g. limitations on maximum power, actuation limits, physical boundaries, etc. The only control architecture which simultaneously caters to all system constraints is a family of controllers called MPC. Most techniques other than MPC resort to ad-hoc methods for dealing with constraints, e.g. anti-windup techniques. Flexible constraint handling capabilities of MPC are a unique feature has inherent robustness qualities, which can be further improved quite easily (Mayne, 2014). MPC technology has found wide application in diverse areas like process, petrochemical, chemical, food processing, manufacturing, aerospace, robotics, etc. It is the standard approach for implementing constrained, multivariable control.

The concept behind this scheme is simple and controller tuning can be achieved by persons not well versed with control engineering, and the concept has evolved to a mature level (Rossiter, 2004). It is a model-based control process, like linear quadratic, pole placement and adaptive control, however MPC has many remarkable features (Mayne et al., 2000), some of which are define by Siddiqui, 2010 as follows:

- A wide variety of processes can be controlled, including non-minimum phase, unstable, time delays and non-linear plants.
- It can be easily extended to multiple input / multiple output (MIMO) plants.
- It is robust to modeling errors to some extent.
- It is relatively easy to tune.
- Process model can be finite impulse response (FIR), step response, transfer function, state space or even non-linear. This is in contrast to linear quadratic (LQ) or pole-placement control.
- Predictive control can cater for process constraints during the controller design itself. It is the most attractive feature of MPC.
- It is an open design framework, i.e. within its broader framework the controller can be designed in a variety of ways, and it can be fused with other control schemes, such as adaptive control.
- Due to its constraint handling, model updatability and inherent robustness it has been proposed and implemented for reconfigurable and fault tolerant control.

2.3.1 Model Predictive Control (MPC)

The MPC is an advanced algorithm being used in chemical and petroleum industries and recently has been applied as well tothe hydropower industry. It uses an internal dynamic model, a history of past control moves, and an optimization cost function over the receding prediction horizon (Arnold, M. and Andersson, G., 2011). Based on the principle, MPC optimizes current timeslot while keeping future timeslots in account. There are various studies where MPC was approached as control scheme for hydropower systems. Nodle et al. (2008) formulated stochastic MPC for the planning of hydro-thermal power where the problems were inflow variation and fluctuations in electric demand. Parameters including prediction horizon and shape of the stochastic programming tree were mentioned as sensitive and analyzed, accordingly. Siebenthal et al. (2005) performed MPC scheme for a cascade of five hydroelectric power plants situated in the river Aare, Switzerland. They analyzed that MPC controller achieved significantly better damping of discharge variations than the local PI controllers used in practice. Furthermore, they mentioned the benefits of coordination between the control actions of the different power plants.

A similar study was done by Setz et al. (2008) where they applied MPC to a cascade of power plants situated along a river in Zurich, Switzerland, that is heavily used for navigation. They derived MPC controller and minimized the changes in turbine discharges within certain tolerance bounds to avoid wearing of equipment and environmental impact. Petrone (2010) performed various tests on the model control using linear and non-linear models for hydropower. His evaluation included performance loss due to imperfect matching of predictive models. He further presented assumptions, possible development of the project and ways to organize the suitable approach of control models. These studies provide an insight for the tuning of MPC that helped in assessing hydropower system at DCK for this study.

2.3.2 Artificial Neural Network (ANN)

Another control scheme is ANN, inspired by biological neural network for its structure and functions. This model is related to principles of pattern recognition, signal processing, and artificial intelligence (Mitchell, 1997). A good understanding of the scheme depends on the choice of model and algorithm. The application of this model employsdiverse fields including robotics, data processing, medicine, classification, regression analysis, and computer numerical control (Yegnanarayana, 2009). The approach is widely used for hydropower system analysis and forecasting. Liang and Hsu (1994) proposed ANN for scheduling hydroelectric generation for 10 hydroplants in Taiwan. Using natural inflow and hourly loads, clustering was done giving four groups on which multilayer feedforward ANN was developed. An algorithm was setup to satisfy all practical constraints. The results from their study showed the ANN approach being faster than conventional dynamic programming approach. More studies have been done to achieve potential results for hydropower.

For Chute-du-Diable hydropower system in northern Quebec (Canada), feed-forward neural network (FNN) was used to assess real-time reservoir inflow forecasting. Two techniques were used to avoid under-fitting or over-fitting on FNN training and enhance generalization performance and the results showed FNN to be effective for improving prediction accuracy (Coulibaly et al., 2000). Likewise, studies demonstrate the comparison of ANN with the traditional approach. Kisi (2004) predicted mean monthly river flow by applying ANN and then compared the results with statistical methods.

Raman and Sunilkumar (1995) modeled the use of ANN in the field of synthetic inflow generation. They developed a neural network and a multivariate autoregressive model for the synthesis and presented a comparison with the statistical methods. An extensive study was done by Cheng et al. (2005) where they applied ANN with feed-forward, back-propagation network structure, and various training algorithms to predict daily and monthly river discharges in Manwan

Reservoir. Their results showed that ANN give more accuracy as compared to conventional time series flow prediction model.

2.4 Hydroelectric Generation in Pakistan

Geographically, Pakistan contains many natural water flow systems that with hydropower potential. This potential land orientation is beneficial for attaining hydroelectricity to meet existing and future demands at minimal cost. There already exist large hydro structures, namely, Terbela dam and Mangla dam generating 3,478 MW and 1120 MW electricity, respectively; however, the small hydropower plants are lucrative in terms of provincial support. Mirza et al. (2008) provides information on the development of hydropower in Pakistan. From the present capacity that is 19,547 MW, 6599 MW comes from hydropower. He further identified the total hydropower potential as approximately 41,722 MW. Following, the public and private sectors are keenly considering hydropower generation projects with a higher response from private entrepreneurs.

Currently in Pakistan, 128 MW of electricity is being produced from small-scale hydropower and 877 MW is under implementation. These projects are operational in all provinces including most in the northern part of country. Table 3 is a list of some small-scale dams in service and their capacities.

Table 3: Small dams of Pakistan and their capacities. Source: Private power and Infrastructure board, 2011)

Location	Dam	Capacity (MW)
Skardu	Shigar dam	1.0
	Gol dam	0.4
	Skardu-I dam	6.96
	Basho-I dam	1.0

Gilgit	Sher Qilla dam	1.1
	Jalalabad dam	1.0
	Gilgit dam	10.63
	Khyber dam	2.0
Jamshoro District	Darawat dam	0.45
Astore	Astore dam	3.1
Chilas	Chilas-I dam	5.62

Alternative Energy and Development Board (AEDB) under the Ministry of Pakistan provides the potential capacity of SHP in Pakistan. Table 4 shows some figures categorized as provincial and regional areas.

Table 4: Potential small-scale hydropower plants (Source: AEDB, 2015)

Location	Number of potential sites	Potential range capacity (MW)	Total potential (MW)
Khyber Pakhtunkhwa	125	0.2 – 32 MW	750
Gilgit-Baltistan	200	0.1 – 38 MW	1300
Azad Jammu & Kashmir	40	0.2 – 40 MW	280
Punjab	300	0.2 – 40 MW	560
Sindh	150	5 – 40 MW	120

Mostly, SHP systems are installed on natural rivers, canals, lakes, and waterfalls. It is convenient to develop a system online sine it requires less infrastructure development and skilled labor.

2.5 Methods

As explained in the introductory chapter, DCK is located in an arid region of southern Pakistan and cannot rely on precipitation for both hydroelectric production and groundwater recharge. Therefore, this hypothesis is being tested where treated municipal wastewater is being brought to small dams through artificially induced streamflow routed through ephemeral streams using standard hydraulic modeling equations. The dam is being programmed to maintain the hydraulic head throughout the day, ensuring maximum hydroelectric production and constant head conditions for artificial groundwater recharge modeling (see Chapter 3). Figure 3 is a graphical representation of this concept.

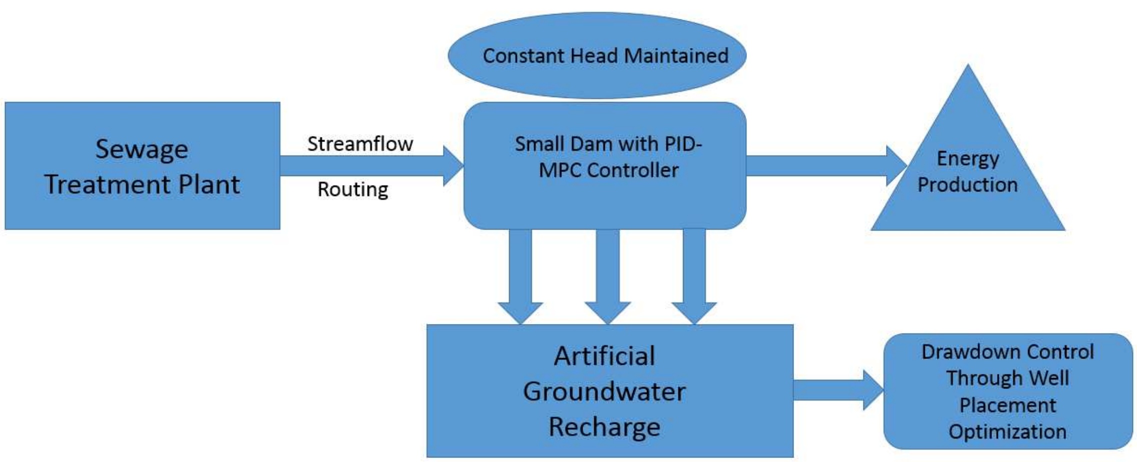

Figure 4: Schematic Diagram for the project.

2.5.1 Identifying Dam Locations

There are two main considerations while identifying locations for these dams; the area that is going to be inundated and the proximity of the dam to a Sewage Treatment Plant. Based on these two considerations, 13 sites were identified for the construction of these small dams in DCK. Stage-Area-Volume analysis were carried out by the AutoCAD Civil 3D Pond-Stage tool in conjunction with the ArcGIS slope analysis tool to determine their height, safe ponding area (so that none of the master planning is disturbed) and maximum probable head keeping a one foot freeboard as a factor of safety. Table 5 shows a list of the dams with their heights whereas Figure 4 shows the ponding areas for these artificial lakes and locations of the STPs.

Table 5: List of Proposed Hydraulic Structures

Hydraulic Structure	Height of the Structure (ft)
Dam 1	32
Dam 2	30
Dam 3	26
Dam 4	29
Dam 5	26
Dam 6	31
Dam 7	30
Dam 8	28
Dam 9	28
Dam 10	29
Dam 11	31
Dam 12	26
Dam 13	28

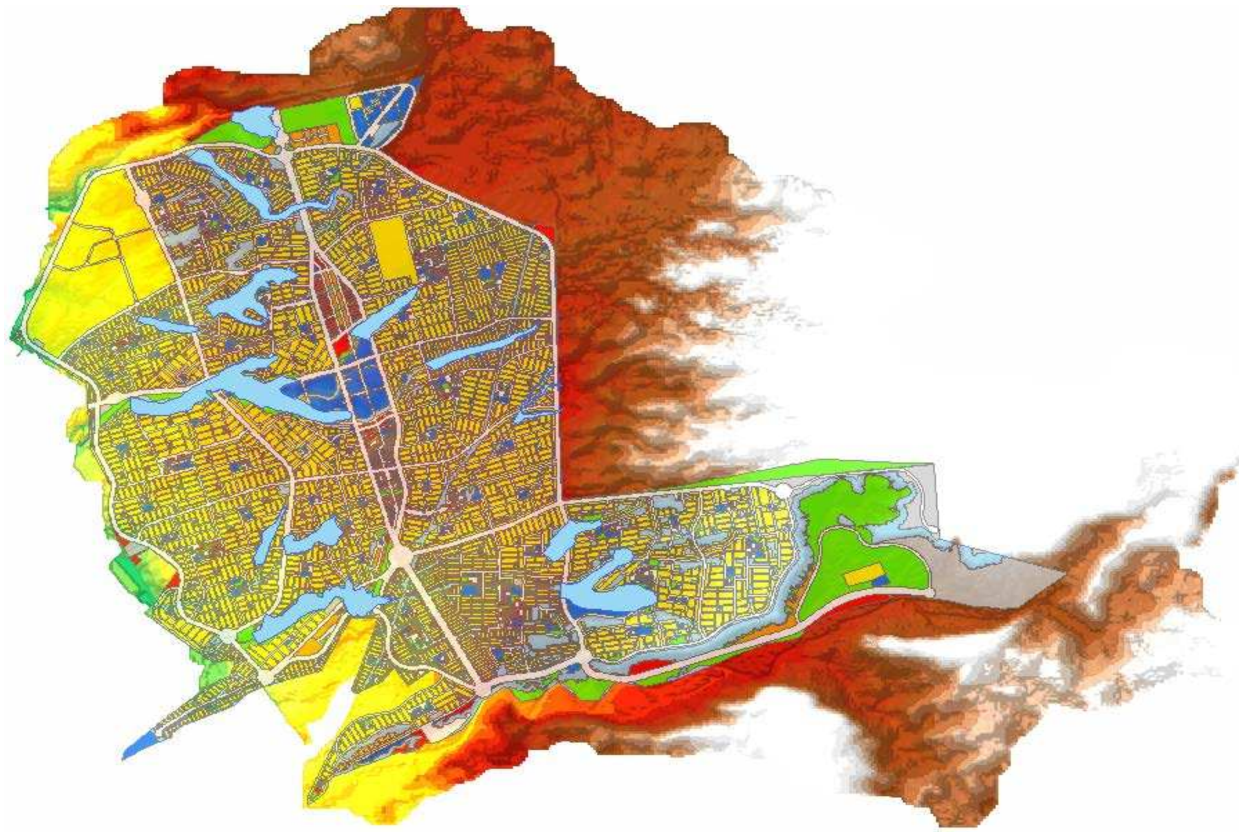

Figure 5: Location of dams and STPs as well as the ponding surfaces.

2.5.2: Routing Streamflow to the Dams

For this study, the streamflow is not being generated from aerial precipitation over an area but from a point source: a sewage treatment plant. The daily amount of expected effluent is known from the plants. However, production is not even throughout the day. In this study, it is assumed that sewage is produced in the same pattern that water demand fluctuates during the day. Qasim et al (2006) defines how the water demand can be broadly divided into peak hours, off peak hours, super peak hours and super-off peak hours. Based on the same concept, our daily sewage production has been divided into these four categories to generate a time series and routed to the dam using standard open channel hydraulics. Losses for evaporation and infiltration have already been subtracted directly. The time series for ET and infiltration are attached as appendices 2 and 3 respectively. There are 11 treatment plants located on the DCK property, with capacities ranging from 6.62MGD down to 0.8 MGD. For explanation, only the modeling of one is described here. Figure 5 shows the distribution of generated flow throughout the day:

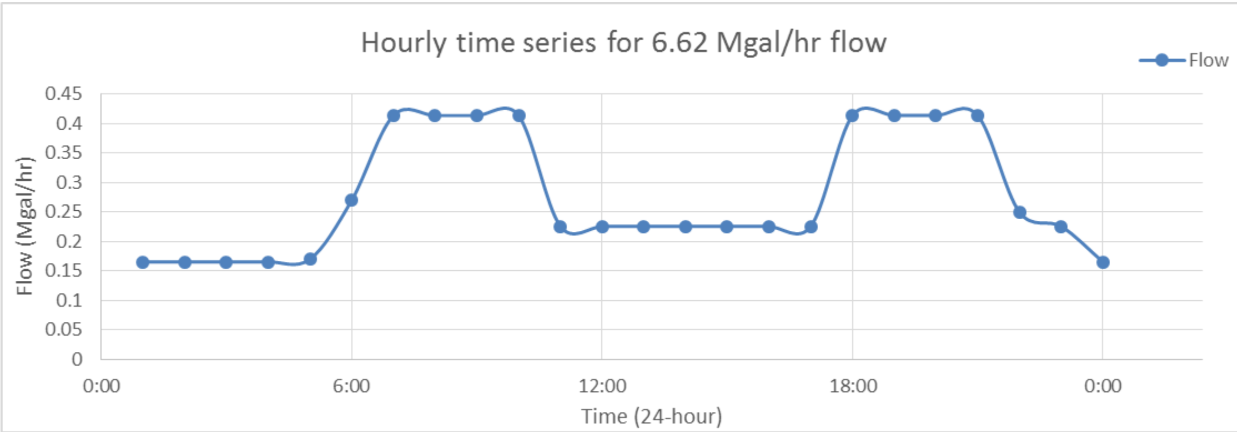

Figure 6: Time Series of Flow generated from STP -1 having capacity of 6.62MGD

Similarly, as per requirement, flows from other STPs have been modelled on the same principles as required to be routed through to the proposed dams.

2.5.3 Designing the Controller

For design purposes, only the process of designing Dam 1 (height of 32 feet, see Table 5) is explained here, with the remaining 12 dams using the same controller with different variables (flow and maximum probable head).

A dam of given dimensions, filled with an inflow $Q_{in}$from wastewater treatment plant, is used for the two purposes of (a) recharging groundwater via seepage through the lake bed by maintaining a certain head h; and (b) maximizing the power production $P$by driving the outflow $Q_{out}$to a turbine. This purpose is served by controlling the gate opening a connecting the penstock to the turbine. The width of the gate b is fixed. Volume change of water $\dot{V}$in the dam is governed by the following differential equation:

$$\dot{V} = Q_{in} - Q_{out} \tag{1}$$

Since,

$$\dot{V} = \frac{dV}{dt} = \frac{d(Ah)}{dt} \quad (2)$$

where is $A = A(h)$is the cross sectional area which varies with head of the dam. Also, the outflow is given by

$$Q_{out} = abC_d\sqrt{2gh} \quad (3)$$

where $g$is acceleration due to gravity and $C_d$is the discharge coefficient given by

$$C_d = 0.611 \left(\frac{h-a}{h+15a}\right)^{0.072} \quad (4)$$

Combining equations (1)-(4), we get

$$\left(A + \frac{dA}{dh}\right) \frac{dh}{dt} = Q_{in} - 0.611\, ab\left(\frac{h-a}{h+15a}\right)^{0.072}\sqrt{2gh} \quad (5)$$

For a constant cross-section area dam ($\frac{dA}{dh} = 0$), differential equation (5) is similar to the model of level of a tank with drain. Using a sampling rate of T_s seconds, (5) can be discretized as

$$h_{k+1} = h_k + \left(Q_{in_k} - 0.611\, ba_k\left(\frac{h_k-a_k}{h_k+15a_k}\right)^{0.072}\sqrt{2gh_k}\right) \times \left(A + \frac{dA}{dh}\right)^{-1} \quad (6)$$

Now the electric power output in megawatts of the turbine is given as

$$P_k = \frac{1}{24000} h_k Q_{out_k} \quad (7)$$

The head is however to be maintained between limits defined by groundwater recharging requirements on the lower side and by physical dimensions on the maximum limit,

$$h_{min} \leq h \leq h_{max} \quad (8)$$

while the manipulatable variable, i.e. gate opening a is also limited by physical constraints

$$a_{min} \leq a \leq a_{max} \quad (9)$$

Moreover, using standard MPC framework, we wish the control objective to be reached by predicting future evolution of (9) for $N_p$time steps using N_C control variations. We want to maximize the power output, which is equivalent to minimizing the following cost function

$$J_k(h, a, N_c, N_p) = \sum_{l=t}^{t+N_p} \quad [5(P_l - P^*)^2 + (h_l - h^*)^2] \tag{10}$$

This cost function represents the desire to produce power near a given set point P^* while maintaining a desired head h^*. The optimal control problem can be written as follows:

At every instant $k \geq 0$, given prediction and control horizons $N_p, N_C \in Z_{\geq 0}$, state h_k, find the optimal control sequence ${a^0}_{k,k+N_C-1}$, which minimizes the finite horizon cost

${a^0}_{k,k+N_C} = argminJk\ (h_{k,k+Np}, a\ \ _{k,k+N_C}, Nc, Np)$(11)

subject to system dynamics (6), state constraints (8) and control constraints (9). Here $\tilde{h}$ denotes the prediction of the trajectory of h using the model (6)-(7).

The loop is closed by implementing only the first element of optimized open loop control (11) at each instant, such that the nonlinear MPC implicit control law becomes

$$\theta_k(h_k) = {u^0}_k(h_k, N_c, N_p) \tag{12}$$

This process is repeated every sampling instant, as illustrated in Figure 6 below.

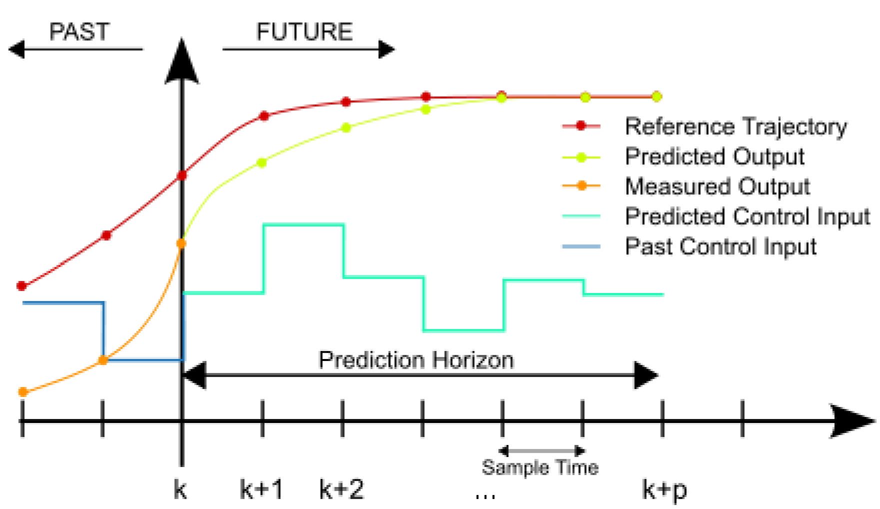

Figure 7: Model Predictive Control concept.

For demonstrating the method, we use the following parameters of the dam model. Width of the gate is taken as b=8 ft (actually two gates of 4ft width each), gravitational acceleration g =

$32.2 ft/s^2$, gate opening is limited to $0 \leq a \leq 6$ft, and the head is to be maintained between $29 \leq h \leq 31$ft (since the height is 32 feet: see Table 5). Change in area of the dam as the head drops, calculated from GIS data, is given in Table 6 below.

Table 6: Change in area of dams with head drops

Head, ft	Surface area, sq ft
30	151238
29	142232
28	132749
27	123731
26	113658
25	105238

Using linear regression, we obtain

$$A(h) = 9300h - 130000, \ \frac{dA}{dh} = 9300 \qquad (13)$$

We choose the sampling interval to be 5 minutes, and the prediction and control horizons to be $N_p = 12$ and $N_c = 5$ respectively. Therefore, we predict and optimize a one-hour trajectory. This sampling time is chosen to lend sufficient time for computation and implementation. Choosing $P^* = 1.5$ MW and $h^* = 30$ft and a constant inflow of $Q_{in} = 900$ cusecs, we obtain the results shown in Figure 7. The results clearly show (in the four figures below) how the system is controlling the operations of the dam through the gates while maintaining the head and achieving maximum possible energy. The controller which already knows the incoming flow as generated from the treatment plant adjusts the configuration of the gates in anticipation to ensure maintenance of head yet producing maximum possible energy by variating outflow.

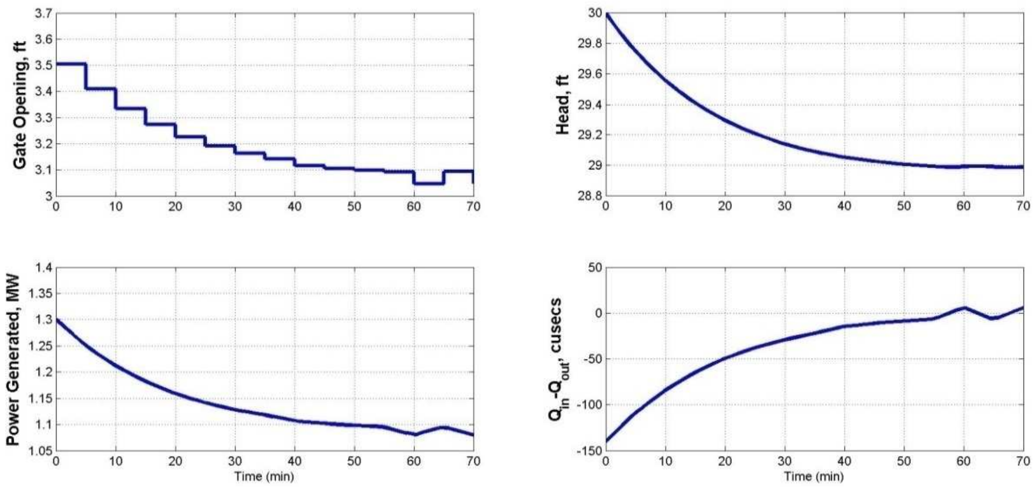

Figure 8: Optimum constrained predictive control of gate opening for ground water recharging dam.

It can be seen that despite the high flow rate and very slow sampling rate, the controller does an excellent job at keeping the head within specified limits and producing optimum power. Compared to a conventional PID controller, the proposed MPC controller exploits the system boundaries and explicitly takes them into account when computing the control action. On a Pentium ® Core-i5 4210U 1.7GHz machine with 4GB RAM, it took less than two minutes to compute the control, which is much less than the 5 minute control interval. This shows the practicality of the proposed controller.

2.6 Results and Discussion of Model Controls

From Section 2.5, it is evident that using the controller to control dam operations can be an accepted method to store and use treated water in arid settings. The head was controlled to within a foot which can be assumed as roughly constant, and maximum electricity was produced. The replication of the same process on the other 12 dams produced the results explained in Table 7.

Table 7: Results of the Hydroelectric Model simulations

Hydraulic Structure	Height of the Structure (ft)	Head Maintained (ft)	Average Electricity (MW)
Dam 1	32	30	1.2
Dam 2	30	28	1.12
Dam 3	26	24	0.96
Dam 4	29	27	1.08
Dam 5	26	24	0.96
Dam 6	31	29	1.16
Dam 7	30	28	1.12
Dam 8	28	26	1.04
Dam 9	28	26	1.04
Dam 10	29	27	1.08
Dam 11	31	29	1.16
Dam 12	26	24	0.96
Dam 13	28	26	1.04

The above table shows 13.92 MW of green and sustainable energy can be produced for this city and a constant head can be maintained, which provides for artificial groundwater recharge through the bed of each artificial lake. The 2015 gazette (attached as Appendix F) from the Karachi Electric (KE) Company shows that one Kwh (Kilo-Watt-Hour) of energy costed 18 cents. Instead of buying at this rate, the green electricity that DCK will be producing at site will save a daily amount of approximately 60,000 USD. The cost of construction of one dam (with 32 ft height) is estimated to be approximately 3 million USD with an O&M cost of $200,000 per year. The dams differ in size but not that much, therefore for the sake of these analysis they are all being considered at the same cost because the other twelve dams are smaller in size so they are assumed to not cost more than Dam 1. That takes the total initial investment to 39 million USD. The total annual O & M cost of the project comes up to be 2.6 million USD. The project should be able to recover its initial investment by the end of year 2 of commissioning.

CHAPTER 3: ARTIFICAL GROUNDATER RECHARGE USING A NETWORK OF ARTIFICAL LAKES

This chapter mostly investigates the previous techniques that have been employed to achieve similar targets. The groundwater modeling tools that are being employed for this research are explained in great detail as well as the construction of the model and assumptions made. This chapter also discusses at length how the water demand has been calculated for the area which is of utmost important as water is a prized commodity in the region and every gallon saved has a cost attached to it. The chapter exhibits results for various well placement scenarios and optimization of drawdown by relocation of wells and fine tuning of pumping rates.

3.1 Introduction: Water Resources in Arid Regions

Having the understanding of accelerating population, per capita water demand, and climate change it has become crucial to attain alternatives to improve and surface water and groundwater resources. Surface water being the easy access has now become highly contaminated by domestic and industrial pollutants and requires higher level of treatment. Groundwater is the other significant source of water supply as it reserves water under the ground, lesser contaminated than surface water, requires less treatment procedures, and provides flow to many lakes and rivers around the world (Spandre, 2009).

In comparison, 30% of world’s freshwater is stored as groundwater, where surface water is only 0.3%. Approximately, 90% of the world's rural population, where there is no water supply network or water company deliverance, use groundwater for drinking and other domestic purposes. The mentioned facts draw attention toward the importance of groundwater and its competence.

Due to the imbalance between recharge and extraction, groundwater conditions have led to exploitation in recent years.

3.2 Groundwater Exploitation in Arid Regions

The exploitation can lead to various transient and perpetual impacts on human lives, land, and economy. Some of the impacts include land subsidence, climate responses, seawater intrusion for coastal aquifers, poor water quality, energy imbalance, and more. A case study by Pacheco Martinez et al. (2013) worked on one of the major geotechnical hazard, land subsidence, due to excess withdrawal of groundwater in the Aguascalientes Valley, Mexico. They mentioned the intensive withdrawal over the years resulting in ground failures such as soil cracks and surface fissures that affected Aguascalientes City and around. They suggest considerable attention from local government and mitigation actions like subsoil investigation and geologic mapping of existing and potential damages. Another study by Changming et al. (2001) mentions the increase in cones of depression in the North China Plain, China, due to excessive discharge of groundwater over the years. The rate of groundwater depletion has been higher beneath the cities making the situation vulnerable to disaster. The major recommendation in this study is groundwater management.

Changes in climate have also been observed as an effect. Zou et al. (2014) performed modeling to study effect of groundwater overexploitation on climate using regional climatic model on Haihe River Basin in Northern China. They investigated using anthropogenic activities and further divided the model into agricultural/irrigation, domestic use, and industrial use. They modeled four sets of 30-year on-line exploitation simulations and one control test. The results showed increase in wetting and cooling effect on the surface and lower troposphere with a consequent decline in the water table. Another study carried out in Belgium estimates sensitivity on groundwater recharge due to climate change for the next millennia. The study used four analogue stations of meteorological time series data (using Global Climatic Model) and HYDRUS 1D for temperature and precipitation and groundwater simulations, respectively. Results showed changes in groundwater recharge varying for colder to warmer regions (Leterme et al. 2012).

Another major resource, energy, is highly complemented with water. A case study in Gujrat state, India elucidates the imbalance of water and energy resulting in damage to groundwater. The study puts forth the situation where due to inadequate surface water, groundwater was accessed to satisfy energy demand. The on-going extraction resulted in water crisis. In addition, the study also provides feasible remedies such as micro water harvesting structures and customer training (Gupta, 2002).

The above mentioned case studies exemplify the limitation of surface water and damage potential due to groundwater exploitation and for this reason, keen research have been carried out for sustainable solutions. From doing various studies, researchers have been successful in proposing strategies of preservation of groundwater in the context of sustainable water management; many of which are now being practiced throughout the world.

3.3 Artificial Groundwater Recharge

One of the most credible alternatives is artificial groundwater recharge. Todd (1959) in his bibliography defines artificial recharge as a mean of increasing the amount of water in aquifers. This method has been implemented for approximately 200 years and used by various countries around the world, such as South Africa, Australia, Hungary, Israel, and the United States. Some of the benefits of artificial recharge are recovery of overexploited aquifers, storage of flood waters and utilization/supply during dry seasons, reduction in subsidence, provision of continuous municipal water, reduction in groundwater salinity around agricultural fields, and reduction in seawater intrusion around coastal areas. However, there can be various disadvantages to the artificial recharge such as discharge of nutrients and micro-pollutants may cause negative long term impacts on soil and aquifer and unless significant amount is being recharged, it may not be economically feasible. The analysis; therefore, is done to attain the best possible analysis beforehand.

Artificial recharge is accomplished using a variety of techniques (natural and artificial) including direct surface recharge, direct subsurface recharge, combination of surface-subsurface methods, and the indirect recharge technique. The direct surface technique involves percolation of water from surface to ground. It is the simplest and widely used application where the construction and operation cost is relatively low. In direct subsurface recharge methods, deeper aquifers are accessed. Water is forced in to the aquifers under pressure using injection wells (or recharge wells). Though this method requires less surface area, it increases construction and maintenance cost and is susceptible to clogging due to impurities. It has been used to dilute coastal aquifers, reduce subsidence, and dispose treated industrial wastewaters (CGWB, 1994).

The combination of surface and subsurface techniques are also used in many countries to meet the recharge and storage needs. Moreover, indirect method is another technique where water is penetrated into ground to modify and/or construct aquifers. However, they provide limited control on water quantity and quality. Groundwater dam is another technique of indirect method where barrier is constructed within the river bed that functions to sustain the storage capacity of aquifer and meet water demands, ultimately. This technique has been executed in India (Helweg and Smith, 1978).

For all the techniques mentioned above, there are prime factors required in designing and operation. Some of the governing factors are area of recharge, length of time, volume of water, and soil-water contact time (Todd, 1980). In addition, hydraulic conductivity, permeability, inflow and outflow are other primary hydraulic factors. Inclusive to the techniques, Scanlon et al. (2002) also mentions about the choosing the technique to quantify the groundwater recharge. They also elucidated on potential recharge and actual recharge and the requirements accordingly.

3.3.1 Artificial Recharge; Case Studies

The European Environment Agency indicates increase in implementation of artificial recharge in countries including Netherlands, Poland, Spain, Belgium, Greece, and Switzerland (Lallana and Krinner, 2001). Using specified techniques artificial recharge of groundwater provides multiple benefits that include increase in water levels, municipal water supply, landscape and/or agricultural purposes, and protection of coastal aquifers from seawater intrusion (Asano, 1985).

In semi-arid and arid regions insufficient rainfall, high rate of evaporation and continuous water consumption disturbs the natural hydrologic balance which results in exploitation of aquifers. Overexploitation eventually decreases groundwater quality and quality effecting the environment and human health. Many of the countries have executed projects involving artificial recharge. Bouri and Dhia (2010) analyzed and experimented artificial recharge for a coastal aquifer in semi-arid region of Tunisia. They tested recharge through wells and surface of dam for 30 years. The results provided evidence of increased piezometer head and improved groundwater quality conditions.

Many countries have used treated wastewater to artificially recharge groundwater. A study by Ouelhazi et al. (2014) investigated the hydrogeological evolution of Korba aquifer, Tunisia, by injecting treated wastewater into the ground. They used 70 piezometers and observation wells throughout the time period to monitor the change in groundwater head. Over the period of 4 years, they observed improvement in water quality and an average increase of 1.5 m in the piezometric levels. Asano and Cotruvo (2004) reviewed reuse of municipal wastewater for recharge focusing health risks. They observed some uncertainties in terms of health and have discussed the criteria by State of California and World Health Organization for the methodology and guidelines toward this practice. Another example of usage of treated wastewater for artificial recharge is for the Wadi aquifer in the Kingdom of Saudi Arabia.

Missimer et al. (2012) demonstrates that the groundwater in the Wadi aquifer is being depleting and increasing in contamination. They evaluated aquifer recharge and recovery (ARR) systems for treating and storing the water to its best quality until needed. Reuse of reclaimed water and restoring the aquifer can beneficial for Saudi Arabia in decreasing desalination costs.

Numerical groundwater flow models have been used in assessing and predicting results of employing artificial recharge systems. Chenani and Mammou (2010) studied groundwater recharge using a combination of a Geographical Information System (GIS) and MODFLOW-2000 (Harbaugh et al. 2000) for a basin in Central Tunisia. Using the interface, they demarcated suitable sites for artificial recharge and estimated the effect of recharge on piezometric levels. They suggested this method as a potential means to manage groundwater conditions, especially for semi-arid and arid regions where the water supply is limited and complex.

Danaein (1997) used MODLFOW to estimate the effect of artificial recharge on a groundwater reservoir in Iran and discussed the positive outcomes. Another case study by Chitsazan and Movahedian (2015) provides calibration and validation of MODFLOW for Gotvard plain in Iran. They performed annual analysis showing effective results of artificial recharge. Moreover, they have mentioned the factors that can possibly reduce the efficiency. Some of the factors are sedimentation, seasonal flooding, and drought.

Katibeh and Hafezi (2004) evaluated the performance of artificial recharge based on drawdown in the city of Bam, Iran. They used MODLFOW to simulate the performance of artificial recharge and concluded that despite the recharge, there drawdown was increasing in future scenario form 10 m to 18 m. They recommended stronger mitigation strategies. Another study in Iran on the aquifer of Goharkooh plain specified the most suitable location for artificial recharge and elucidated positive artificial recharge results from the model (Rezaei and Sargezi, 2009). In a case study by Tabari et al. (2012) used monthly groundwater level data to model recharge for 14 year-period. This strategy aimed to avoid floodwater spreading and utilize it for groundwater recharge. The study focused spatial distribution along with changes in hydraulic and hydrologic parameters and results showed effective control of flood with efficient recharge method. These studies provide clear evidence toward affirmative effect of artificial recharge approach for arid regions.

The technique of artificial recharge has also been assessed for its benefits including water supply and irrigation. A study by Mirlas et al. (2015) assessed the strategy for an agricultural area and neighboring rural settlement in Kazakhstan. Using MODLFOW, simulations show balance in drawdown and recharge conditions that are suitable to restore the groundwater for provision of water supply to settlement and agricultural fields. Moreover, the groundwater mound is estimated to serve a barrier for contamination from irrigation fields. Lu et al. (2011) performed groundwater modeling at five sites in the Heibei Plain, China, to simulate the recharge influence on irrigation and water table. HYDRUS 1-D was calibrated for each site using climatic data, groundwater levels, and soil moisture. Henceforth, the results for each site showed variance in spatial and temporal patterns for each site concluding that time-lags are to be considered for every water table depth and recharge study.

3.4 Estimating DCK's Water Demand

Annual average, maximum day and peak hour demands are important for planning and designing of municipal water supply systems. These variations are conveniently expressed as ratio to the mean average daily flow. These ratios vary greatly for different cities; therefore a careful study for each city must be made from the past data to develop these fluctuations.

The residential or domestic water demand is the portion of the municipal water supply that is used in homes. Residential water use may vary greatly depending upon the environmental conditions and social norms of the society. This use is also highly dependent on the environment and socio-economic conditions. It includes toilet flush, cooking, drinking, washing, bathing, watering lawn, and other uses. According to McGhee (1991), the average residential water demand varies from 25 GPCD (Gallons Per Capita per Day) to 120 GPCD, while most commonly used numbers are 50 – 80 GPCD for urban areas in the world. Karachi Water and Sewerage Board (KWSB) is the main authority responsible for supply and distribution of water to Karachi city. It has assessed an average domestic water requirement water requirement as 54 gallons per capita per day for the whole city. It projects the water demand for future expansions of Karachi City on the same basis. As per provincial public health engineering departments, this value varies with the planned population of the settlement. It increases with the increase in population of the city. Qasim et al (2006) derived a worthy comparison of water demand per capita averages of different cities around the world; Tokyo – Japan (77 GPCD), New York – USA (161 GPCD), Colombo – Sri Lanka (60 GPCD) and Mumbai – India (43 GPCD). Qasim et al also made a breakdown of how water is consumed in a residential unit with respect to various activities globally. Ihsanullah (2009) made a similar breakdown for a typical 500 square yard residential plot in Karachi. Most of the persons who have purchased plots of land for residential or commercial purposes in the DCK are likely to belong to an affluent section of the population of Pakistan. The houses or dwellings constructed by them are likely to be of superior quality depicting a higher standard of living, which will require a larger amount of per capita water consumption. It is anticipated that this may necessitate a higher per capita water supply to be provided for the water supply system for this area in comparison to the general average provision normally adopted by KWSB for their water supply projects.

Commercial establishments include hotels, office buildings, shopping centers, service stations, entertainment centers and the like. The commercial water demand depends on the type and number of commercial establishments. Generally, the commercial water demand is about 10-20 percent of the total water demand. Based on floor area ratio (FAR), the typical value ranges from 0.1 to 0.4 gallons/ft^2.The average water demand as per planned commercial plots for DCK is given in the Table 8:

Table 8: Average Water Demand for Commercial Plots in DCK

Commercial Planning	Average Water Requirement (GPD)
Market (variable plot size with single floor)	350 – 6,500
C1 (125 sq.yds plot with 1:4 FAR)	675
C2 (200 sq.yds plot with 1:4.5 FAR)	1,215
C2.5 (250 sq.yds plot with 1:4.5 FAR)	1,519
C3 (300 sq.yds plot with 1:5 FAR)	2,025
C5 (500 sq.yds plot with 1:5 FAR)	3,375
C10 (1000 sq.yds plot with 1:5.5 FAR)	7,425
C20 (2000 sq.yds plot with 1:5.5 FAR)	14,850
Mall Zone (variable plot size with 1:5.5 FAR)	41,800 – 758,000

Gas Station(Average daily 50 Carwash with 20 Employees)	3,000
M1 (Two floors in Mixed use of 125 sq.yds plot)	338
M2 (Two floors in Mixed use of 200 sq.yds plot)	486
M2.5 (Two floors in Mixed use of 250 sq.yds plot)	608
M3 (Two floors in Mixed use of 300 sq.yds plot)	648
M5 (Two floors in Mixed use of 500 sq.yds plot)	1,013
M10 (Two floors in Mixed use of 1000 sq.yds plot)	1,890
M20 (Two floors in Mixed use of 2000 sq.yds plot)	3,645

Water used in public buildings (Community Hall, Marriage Hall, Officers Mess, Club, Culture & Art, schools etc) as well as in public services (including fire protection, street washing, and landscape irrigation) is considered public water use. Usually public water use accounts for 5-10 percent of total municipal water demand. As per planning of DCK, the average water demand for different public buildings is given in Table 9. The water demand may become more precise when actual detailed building structures are planned.

Table 9: Average Water Demand for Public Water use in DCK

Land use	Average Water Requirement
Community Hall, Marriage Hall, Officers Mess, Club, Culture & Art	0.15 gallon per sq.ft per day

Mosque	0.5	gallon per sq.ft per day
Hospital	250	gallon per bed
DCK Clinics	0.15	gallons per sq.ft per day of covered area
College/School (day scholar, without boarding)	17	per student
University	34	gallons per student per day
Golf Club	0.02	gallons per sq.ft per day
Lake-view Park	0.02	gallons per sq.ft per day
Parks and Playground	0.01	gallons per sq.ft per day
Theme Park	0.01	gallons per sq.ft per day
Nursery	0.02	gallons per sq.ft per day
Road Landscaping	0.006	gallons per sq.ft of ROW per day

3.5 DCK Water Requirement

The total average water requirement when DCK will be fully developed is expected to reach up to 45.5 MGD (as assessed value is 45,551,801 GPD). The average water demand per unit area is around 0.09 gallons/ft^2. For the assessment of this demand detailed land use planning was used. The whole DCK Sector-1 is reserved as a residential housing sector for the Army and it will be planned later in detail. However, as a residential sector its average demand is around 1.6 MGD. Without Sector 1, the assessed water demand is around 43.9 MGD. Table 10 shows the distribution of water requirement in DCK excluding the Sector 1:

Table 10: Average Water Demand of Fully Developed DCK

#	Land Use	Average Water Demand	
		(GPD)	%
1	Residential	17,254,806	39
2	Commercial	7,661,986	17
3	Mixed Use (Residential + Commercial)	7,358,539	17
4	Amenities	3,928,420	9
5	Recreation	743,475	2
6	Utilities	5,408,578	12
7	Agriculture and Preserved Open Spaces	544,949	1
8	Transportation	1,044,649	2
	Total	43,945,402	100

In the whole water requirement, the biggest share is of residential use (39% of water demand) which occupies the major share of land in DCK (37.53% of land use). However water demand per square foot for residential area is quite low (0.09 gallons/ft^2).

The further distribution of water requirement on various sizes of plots in DCK is given in Table 11. Within residential land use, more than 50% water will be required for R-5 (i.e. plots of 500 square yards).

Table 11: Distribution of Water Requirement in Residential Land use

#	Type	Water Requirement	
		(GPD)	%

1	R-1 (125 sq.yds)	1,997,622	11.6
2	R-2 (200 sq.yds)	2,133,362	12.4
3	R-2.5 (250 sq.yds)	66,177	0.4
4	R-3 (300 sq.yds)	2,147,521	12.4
5	R-5 (500 sq.yds)	9,571,041	55.5
6	R-10 (1000 sq.yds)	1,122,393	6.5
7	R-20 (2000 sq.yds)	216,691	1.3
	Total	17,254,807	100.0

In DCK, almost 2% area is designated for commercial plots of various sizes in community centers and in central business district. However, some large plots with higher FAR are planned as Mall Zones that will require the largest commercial water demand. The overall water demand for commercial plots is 7.6 MGD.

Detailed distribution of average water requirement in commercial land use is given in Table 12.

Table 12: Distribution of Water Requirement in Commercial Land use

#	Type	Water Requirement	
		(GPD)	%
1	C1 (125 sq.yds plot)	18,070	0.24
2	C2 (200 sq.yds plot)	1,476,064	19.26
3	C2.5 (250 sq.yds plot)	33,649	0.44
4	C3 (300 sq.yds plot)	880,046	11.49
5	C5 (500 sq.yds plot)	1,101,707	14.38

6	C10 (1000 sq.yds plot)	757,702	9.89
7	C20 (2000 sq.yds plot)	108,326	1.41
8	Other commercial plot sizes	223,885	2.92
9	Gas Station	42,000	0.55
10	Mall Zone	2,964,434	38.69
11	Market	56,103	0.73
	Total	7,661,986	100.00

Its vertical height (FAR) and horizontal size (plot size) with density of water users (population) are the major water demand controlling factors. In DCK, the second highest water requirement will be for it, which is almost 7.3 MGD (17% of the total water demand). Although most of the mixed use plots in community centers (C3, C4 and C5) are comparatively small in size, however large designated plots in CBD and South Zones are the major consumers due to their sizes and respective FAR. Further distribution of water demand with respect to plot size is given in Table 13:

Table 13: Distribution of Water Requirement in Mixed use (Residential + Commercial)

#	Type	Water Requirement	
		(GPD)	%
1	M1 (125 sq.yds plot)	2,093	0.03
2	M2 (200 sq.yds plot)	455,256	6.19
3	M2.5 (250 sq.yds plot)	23,606	0.32
4	M3 (300 sq.yds plot)	890,978	12.11
5	M5 (500 sq.yds plot)	646,031	8.78

6	M10 (100 sq.yds plot)	204,626	2.78
7	M20 (2000 sq.yds plot)	2,560,893	34.80
8	Other plot sizes of Mixed use	2,575,056	34.99
	Total	7,358,539	100

A long list of required amenities is planned in DCK that covers around 4.7% (543 acres) of the DCK area. It consists mainly of Educational Institutions, public buildings, religious centers, health facilities and some reserved amenity spaces for future requirements. The average water requirement per unit area for amenities is 0.16 gallons/ft^2 with overall requirement of 3.9 MGD.

Educational institutions are the main water consuming units due to the strength in numbers of attracted users (students). Further detail of average water requirement is given in Table 14:

Table 14: Distribution of Water Requirement in Planned Amenities

#	Type	Water Requirement	
		(GPD)	%
1	Educational Institutions	2,296,387	58.46
2	Public Buildings	353,193	8.99
3	Religious Buildings	676,191	17.21
4	Health Facilities	548,877	13.97
5	Reserved Amenities	53,771	1.37
	Total	3,928,420	100.00

Approximately 7.6% of DCK land is designated for recreational purpose. This share will not only serve the local recreational demand but also cater for the regional requirement of high quality amusement and sports facilities such as Theme Parks, Golf Courses and specialized sports centers. The main requirement of water in recreational use is related to the landscaping and green spaces development. It requires around 5 MGD (8.7% of the total water requirement of DCK). Its per unit ground area water requirement is comparatively low, which is 0.13 gallons/ft^2. Further distribution of water requirement in planned amenities is given in Table 15.

Table 15: Distribution of Water Requirement in Recreational use

#	Type	Water Requirement	
		(GPD)	%
1	Theme Park	313,725	42.20
2	Golf Club	134,063	18.03
3	Parks and Playground	185,718	24.98
4	Sports & Recreational Parks	76,479	10.29
5	Nursery	33,490	4.50
	Total	743,475	100.00

As the first sustainable city of Pakistan, DCK is planned in a great comprehensive manner which is going to minimize its dependency on external resources. In this connection, its entire energy requirement is planned to produce at site with the blend of renewable and conventional energy sources. Similarly, water requirement is going to be fulfilled by optimizing the conjunctive use of alternate water supply with regular water supply from the bulk water source. Almost 376 acres of land (3.24%) has been designated for exclusive use of utilities to manage it in a sustainable manner. The overall average daily water requirement for the complete city is around 5.4mgd, with the lion's share for various processes involved in conventional power generation (i.e. 4.2 MGD –

79% of total utilities requirement). Its per unit area requirement is higher (0.34 gallons/ft^2). Further details are given in Table 16.

Table 16: Distribution of Water Requirement for Utilities

#	Type	Water Requirement (gpd)	%
1	Conventional Energy Park	4,274,683	79.04
2	Wind Energy Park	592,956	10.96
3	Solar Energy Park	336,229	6.22
4	Sewage Treatment Plant	68,284	1.26
5	Unclassified Utility	61,001	1.13
6	Electric Installation	47,927	0.89
7	Water Pumping Station	18,552	0.34
8	ICT and Security	8,946	0.17
	Total	5,408,578	100.00

DCK focuses on an urban green environment with a good architectural design landscaping encompassing multi-parameters. Fruits, flower, flower colors, fragrance, birds ‘attraction and water requirement are the main selection parameters for soft landscaping. In this connection, other than recreational facilities, transpiration corridors are also planned with soft landscaping. Transportation corridors cover around 4,004 acres (34.5% of DCK land) with different Right of Ways (ROWs) and road categories. The water requirement for various transportation usewill be around 1 MGD with highest share at streets and pedestrian pathways. The details are given in Table 17.

Table 17: Distribution of Water Requirement in Transportation use

#	Type	Water Requirement	
		(GPD)	%
1	Streets and Pedestrian way	619,479	59.3
2	Arterial	233,882	22.4
3	Collector	150,405	14.4
4	Parking and Terminals	40,883	3.9
	Total	1,044,649	100.0

There are some reserved areas within DCK which are required to be kept green that includes ROW of oil and gas pipelines passing through the DCK, reserved areas for agriculture and some open and green spaces along the storm water drainage corridors. These areas will require around 0.5 MGD with low water requirement per unit area (approx. 0.01 gallons/ft^2). The details are given in Table 18.

Table 18: Distribution of Water Requirement in Agriculture& Reserved Spaces

#	Type	Water Requirement	
		(GPD)	%
1	Open & Green Space	505,491	92.76
2	Gas Pipe Line	32,360	5.94
3	Oil Pipe Line	6,797	1.25
4	Reserved Areas for Agriculture	301	0.06
	Total	544,949	100.0

3.6 Spatial Distribution of Water Demand

Table 19 shows the zone-wise distribution of required water, in which more than half (60%) of the water demand is required in residential sectors with a comparatively low water requirement per unit area (0.07 gallons/ft^2). Second largest requirement of water is in the South Zone that is almost 19.2% of the total demand with comparatively higher value per unit area (0.15 gallons/ft^2). The highest water requirement per unit area is in City Gateway and Downtown (i.e. 0.22 gallons/ft^2) that covers around 16% of the land. Further sector wise water requirement is given in Table 20 and Figure 8. Figure 9 shows the sector wise split of water demand according to the land use planning.

Table 19: Zonal Distribution of Water Requirement

Planning Zone	Average Water Requirement		
	(gpd)	%	g/ft^2
DCK Sectors	27,452,937	60.27	0.07
South Zone	8,767,139	19.25	0.15
Downtown and City Gateway	7,338,263	16.11	0.22
Misc. Areas	1,993,462	4.38	0.04
Total	45,551,801	100.00	0.09

Table 20: Sector wise of Water Requirement

#	Sectors	Water Demand (WD)	Area (Acres)	WD/ft^2
1	Sector 1	1,611,477	520.6	0.07

2	Sector 2	1,712,654	594.4	0.07
3	Sector 3	1,042,493	310.3	0.08
4	Sector 4	1,115,381	360.3	0.07
5	Sector 5	1,348,453	482.6	0.06
6	Sector 6	2,949,705	822.2	0.08
7	Sector 7	1,832,853	626.7	0.07
8	Sector 8	466,956	249.6	0.04
9	Sector 9	2,157,922	604.2	0.08
10	Sector 10	1,811,381	540.9	0.08
11	Sector 11	1,828,548	626.2	0.07
12	Sector 12	1,804,993	503.0	0.08
13	Sector 13	1,813,055	446.8	0.09
14	Sector 14	3,465,420	654.8	0.12
15	Sector 15	1,657,600	696.7	0.05
16	Sector 16	834,045	383.0	0.05
17	Gateway	2,419,675	291.9	0.19
18	Downtown	4,918,588	455.3	0.25
19	Southern Zone	8,767,139	1,323.4	0.15
20	Misc. Areas	1,993,462	1,095.0	0.04

*Sector 1 is not planned in detail. Its water demand has been assessed on the basis of average sector demand per unit area

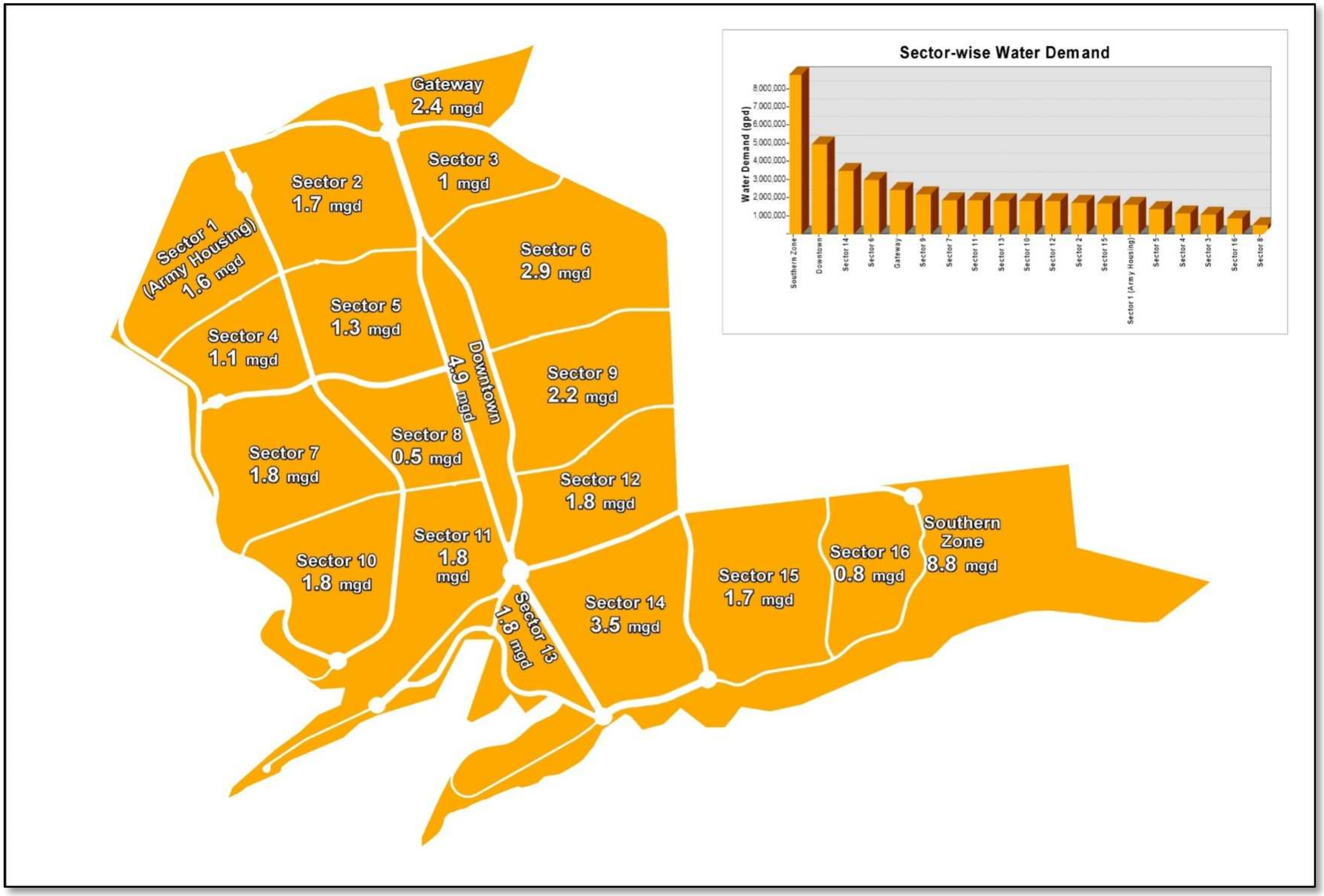

Figure 9: Sector wise distribution of water requirement in DCK

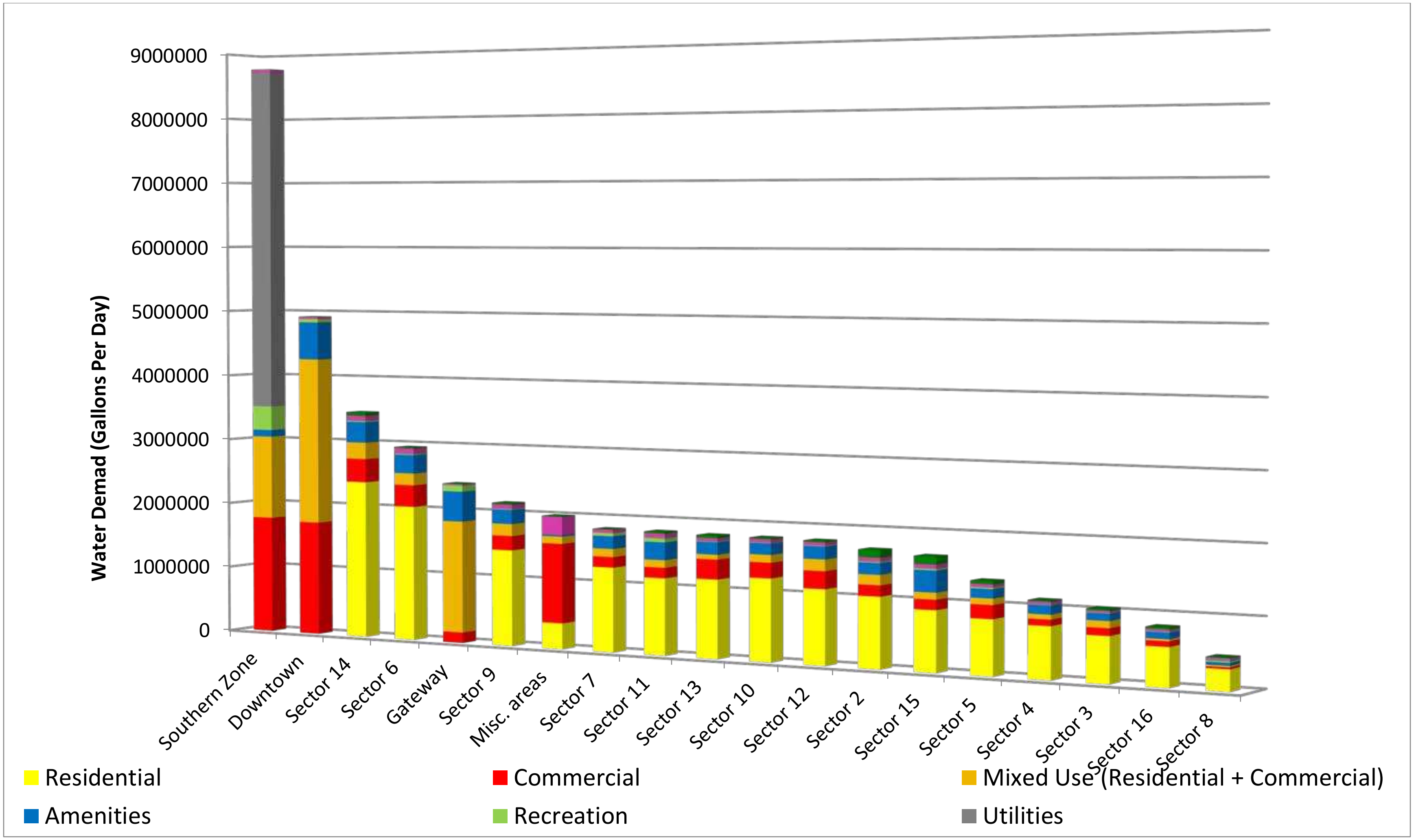

Figure 10: Water Demand Distribution with respect to Land-Use.

3.7 Temporal Water Demand

As per DCK development strategy, it is going to develop in three phases i.e. Short Term (2012-2015), Mid Term (2015-2020) and Long Term (2020-2030) (see Table 21, Figure 10 and Figure 11). By considering the development plans it is estimated that for Short Term Development, 11.1 mg water will be required which will serve the complete demand of the short term planned area. However, its water resource development will be tricky in the beginning. Just for the short term development plan, average annual water resource development requirement is around 3.7 MG per year. However, actual use of water will be started with the occupancy and use of land for the respective purpose and migration trends. It is estimated that in the initial stage, water will only be required for construction purposes and only 10 to 15% of water use will be developed for DCK Camp Site etc. till the end of short term development plan. It is envisioned to develop around 1.5 to 3 MG of water resources by the end of short term development plan to fulfil the needs.

Table 21: Water requirement with respect to DCK development phases

Development Plans	Water Requirement After Completion (MGD)	Yearly Water Resource Development Requirement (MGD)
Short Term (2012-2015)	11.1	3.7
Mid Term (2015-2020)	20.7	4.14
Long Term (2020-2030)	11.5	1.15
Sector 1	1.6	1.6

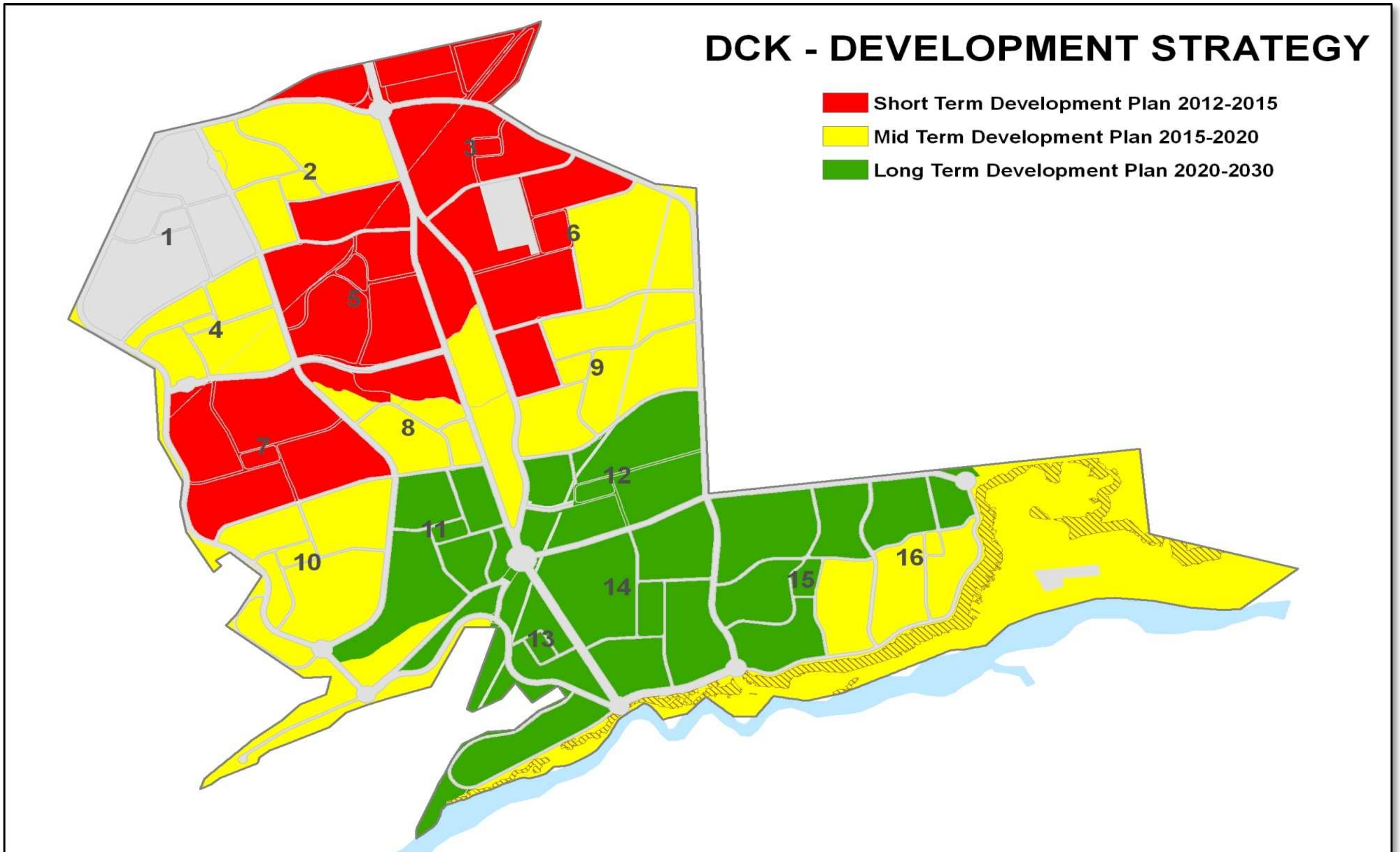

Figure 11: DCK Development Strategy

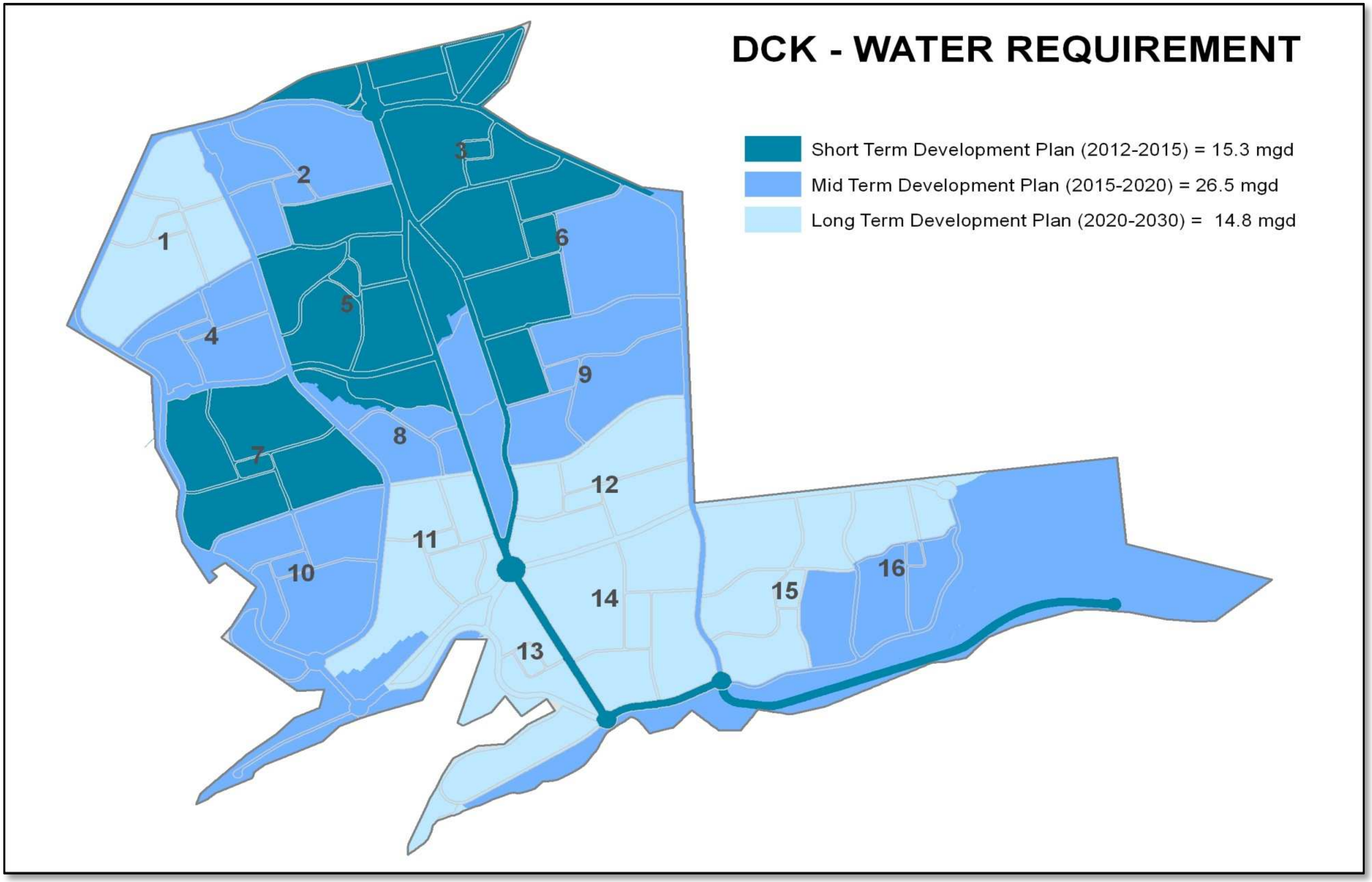

Figure 12: DCK Water Requirement as per Development Plans.

3.8 Methods

This section provides comprehensive method applied to attain results for groundwater recharge and pumping optimization. Extensive literature is done that provided an insight on the existing work done by various other relevant researchers. Having done that, existing dataset is analyzed along with collection of other required data. Various models are studied where MODFLOW is chosen based on availability of data and accessibility of model. Figure 12 is a graphical representation of the steps carried out for this research.

The model is constructed where grid cells, initial head, boundary conditions, and other aquifer properties are defined. Following the pre-processing of model, hydraulic properties are estimated and model is set for simulations. Groundwater modeling for this study includes two components, artificial recharge and pumping. Modeling for artificial recharge is done for the next 20 years. Thereafter, modeling is performed for drawdown i.e. pumping through wells. Four scenarios are created having variation in wells placement and simulations are performed to estimate the optimization of pumping location. This chapter also includes cost-benefit analysis of recharge and pumping.

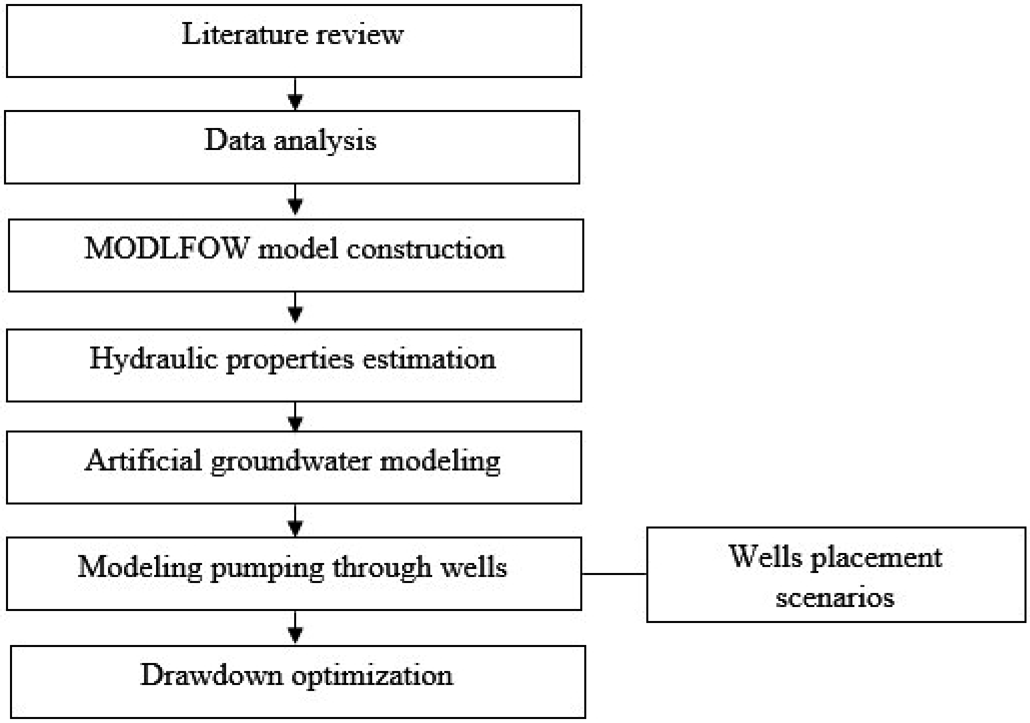

Figure 13: Methodology flowchart for groundwater modeling

3.9 Estimating Volume of Artificial Recharge

This section describes the construction and use of a 3D MODFLOW model to estimate the quantity of recharge that can be expected when the artificial lakes are constructed. When the initial data collection drive was being planned for the DCK, the idea of the artificial groundwater recharge project had yet not been conceived; therefore, data was not specifically collected for this purpose. A groundwater exploratory survey drilled 13 boreholes out of which 6 bores were successful and some pumping tests were conducted on the same day, the wells were not continuously monitored during consumption hence there is no available data to calibrate it against. A DEM (Digital Elevation Model) was developed from the physical data points taken for x,y and z coordinates at every 1x1 meter. For surface hydrology, this DEM was merged with ASTER 30M GDEM (accessed on July 12th, 2014) to complete the watershed boundaries which also helps in our groundwater modeling exercise as it defines the regional flow divide. For this modeling work, Meters and Days were chosen as the units for distance and time respectively.

3.10 MODFLOW Model Construction

3.10.1 Defining the Grid and Active Cells

This study aims to define groundwater head and flow at the regional scale of the DCK area. Thus, a coarse grid of 100m x 100m cells was used, resulting in 94 rows and 122 columns to cover the area of around 8 million m^2. The merged DEM is used as the top of the grid, and the coverage of the model top extending to all cells were used to differentiate the active cells from the inactive cells. The grid was divided vertically into 3 layers based on the borelog data having depths of 50, 100 and 150 meters respectively for a total depth of 300 meters. Figure 13 shows the model grid with the active cells as defined using the model's Graphical User Interface (GUI) ModelMuse. Figure 13 shows the grid for the study area and the active cells shown in blue whereas the inactive cells are displayed as red whereas the contour elevations are displayed in meters.

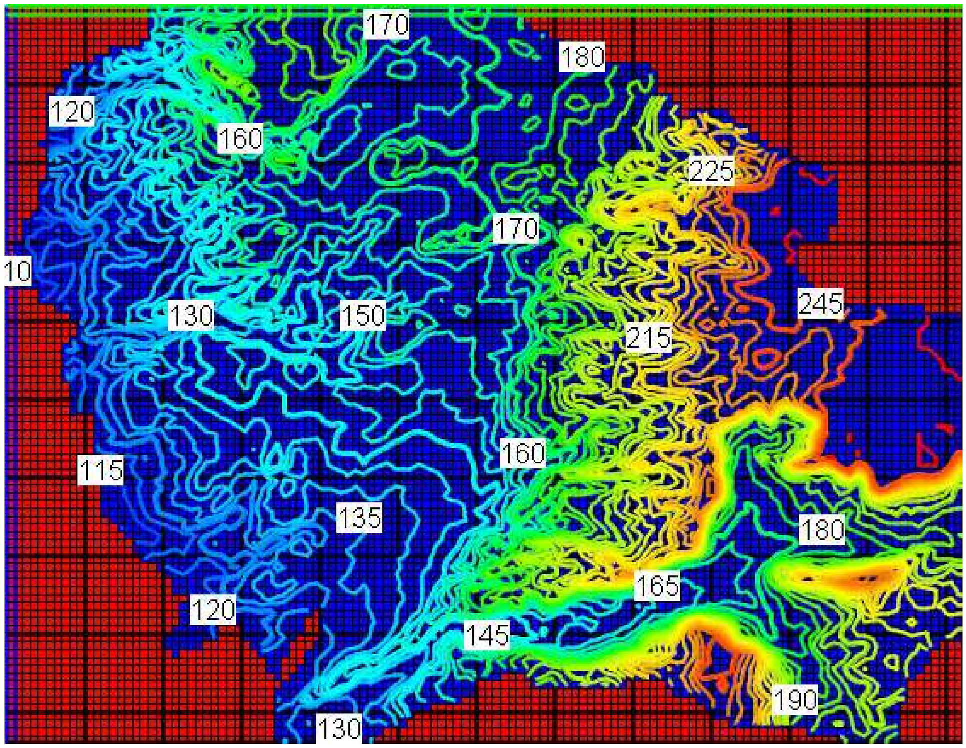

Figure 14: Model Grid with model top contours and active cells

The Layer Property Flow (LPF) package is used to simulate the flow and define the aquifer properties. The Geometric Multigrid (GMG) Package solver is used as it is preferable for larger grids.

3.10.2 Defining Initial Groundwater Head

There are 6 data points scattered along the mid and mid-west region of DCK from north to south which give observed measurements of depth to water table; hence, an average of these depths to water table was used as the model initial head. This resulted in an initial head of approximately 21 m below the ground surface.

3.10.3 Assigning Aquifer Properties

Due to unreliability of the pumping tests, the hydraulic conductivities were calculated as weighted averages of the materials logged during boring at the 6 drilling sites. The typical values for these materials were taken from Heath (1983) as well as from Domenico and Schwartz (1990). Analyzing the bore logs, the vertical grid was divided into 3 layers.

The top layer which covers the strata between the ground surface to 50 meters below consists mainly of fractured limestone coupled with silty sand, sandstone and very little sandy marl. A weighted average value of 0.162 m/day was assigned as horizontal hydraulic conductivity (*K*), with vertical hydraulic conductivity equal to $1/10^{th}$ of horizontal *K*. The middle layer covers the strata between 50 meters and 150 meters below the surface that is a depth of 100 meters. It mainly consists of clayey sandstone and marl limestone coupled with silty sandstone. A weighted average value of 0.094 m/day was assigned to horizontal*K*, and vertical *K* was $1/10^{th}$ of horizontal *K*. The bottomlayer consists mostly of sandstone and silty sand with traces of clayey materials like clayey sandstone and marl. It extends 150 meters below the middle layer. A weighted average value of 0.221 m/day was assigned to horizontal*K*, and vertical *K* was $1/10^{th}$ of horizontal *K*.

For Specific yield, values presented by Morris and Johnson (1967) were used. Values of 15%, 10% and 20% were assigned to the top, middle and bottom layers respectively with respect to the weighted averages of their material compositions. Bore logs and water table data are attached as Appendix D.

3.10.4 Boundary Conditions

The boundary condition packages that are being run for this simulation are the Flow and Head Boundary (FHB) Package, plus the source Reservoir (RES) Package and the sink Well (WEL) Package.

3.10.4.1 Source: Flow and Head Boundaries

The groundwater aquifer in DCK is not confined to the administrative or watershed boundaries. To cater for the flow entering and leaving from the boundaries, a specified head boundary is applied to the boundaries using the extrapolation of the ratio between the ground surface elevation and the water table. Hence, specified head boundaries of 48m below ground surface to the eastern and south eastern boundaries, 38m below ground surface to the north eastern boundary, 50m below ground surface to the southern boundary, 30m below ground surface to the south western boundary, 25m below ground surface to the northern boundary and 15m below ground surface to the western boundary are applied.

Figure 14 shows one such example of extrapolation. This extrapolation is for the eastern boundary. Data from two boreholes, one located on the western boundary and the other located in the Midwest 2800m away from each other are extrapolated all the way to the eastern boundary 8500m away with the known surface elevation.

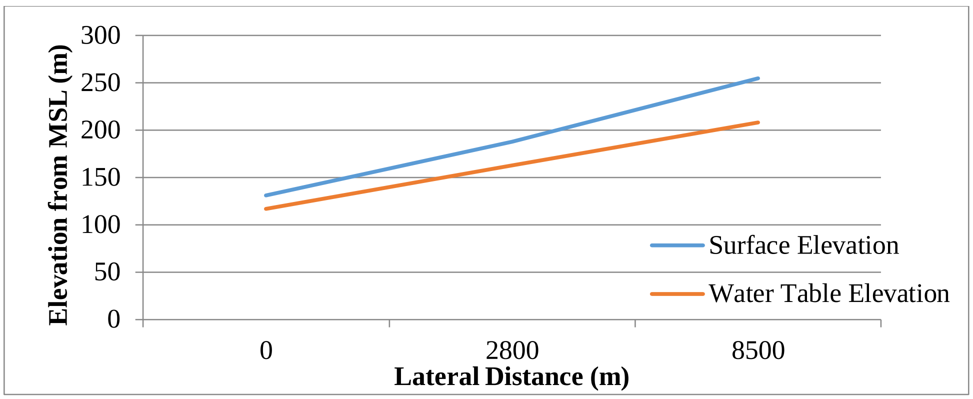

Figure 15: Extrapolation of head boundary for the eastern edge

The flow and head conditions are considered as both, source and sink. The upstream is source whereas, downstream is considered sink.

3.10.4.2 Source: Reservoir Package

The purpose of this package is to simulate the leakage between the reservoir and the aquifer as the changing stage of the reservoir makes the lake contract and expand. However, the main the difference between the reservoir package and the lake package is the fact that the interaction between the aquifer and the lake does not affect the head in the reservoir. Since, we are applying

Model Predictive Control (MPC) and Feedback Control system to the dams in order to maintain the head in presence of guaranteed flows from the Sewerage Treatment Plants (STPs). For MODFLOW purposes, the lakes were marked after the locations and maximum heads were carefully selected on the criterion of i) being in close proximity plus downstream of treatment plants and ii) ensuring that no city planning is submerged and the ponded areas do not extend beyond the DCK administrative boundary. The lakes are marked using the slope analysis tool in ArcGIS that gives the extent of the ponded surface with respect to the given head. Once the polygons for the ponded areas are marked, they are exported as Shape Files to Model Muse where they are assigned as objects to be used by the RES package for the enclosed cells. All the enclosed values are assigned a head value as defined and the difference between the defined head and the surface elevation is taken as the ponded volume over the aerial extent. The equations used in this package calculated seepage leakage from the reservoir (Fenske et al. 1996).

$$Q_{res} = CRES\,(h_{res} - h_{gw}) \tag{14}$$

$$CRES = \frac{Hcres \times DELC\,(I) \times (J)}{Rb_{thick}} \tag{15}$$

Where,

HCres is vertical hydraulic conductivity of the reservoir bed (L/T), *DELC(I)* is width of model row I (L), *DELR*(J) is width of model column J (L), Rbthick is thickness of the reservoir-bed sediments (L), Q_{res} is the leakage from the reservoir (L^3 /T), h_{res} is reservoir stage (L), and h_{gw} is the ground-water head (L).

Figure 15 depicts the position of the 13 lakes with respect to the town planning and the DEM which are co-incidentally also the active cells.

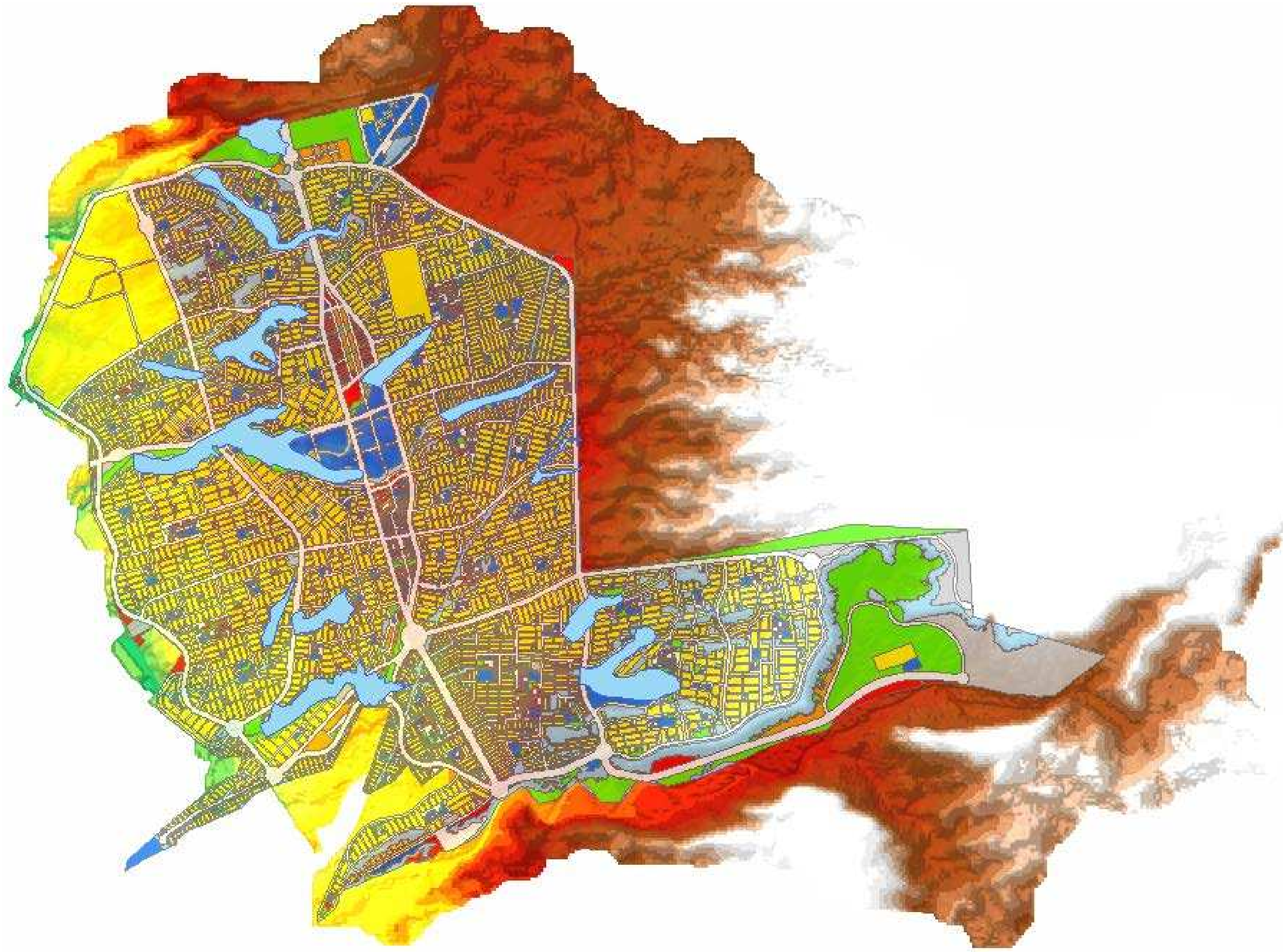

Figure 16: ArcGIS image showing the lakes, town-planning and DEM overlaid.

The reservoir bed thickness was specified as 1 m, whereas the horizontal *K* for the top layer of the grid was selected as the reservoir hydraulic conductivity.

3.10.4.3 Sink: Well Package

The well package is used to simulate a specified flux out of a selected cell. The WEL package in this research is used only in the second half once the recharge has been modelled and calculated. Harbaugh et al.(2000) defines the working of the WEL package in detail. After initial model simulations are run to determine the available groundwater due to artificial recharge, pumping is simulated to determine the optimal placement of 50 production wells in the DCK area. The optimal placement is based on minimizing groundwater head drawdown.

3.11 Defining the Stress Periods

The stress periods for this simulation have been defined based on the developmental strategy described in section 2.2.3. DCK is to be developed in three phases; short-term plan, mid-term plan and long term plans. Therefore, the population will increase in the same pattern, hence, the water resources need to be developed on the same criteria. The complete simulation is divided into 4 stress periods where the first stress period is a one year steady state simulation with no lakes putting water into the system and no wells taking water out of the system. The other 3 stress periods are divided as per the development strategy into 4, 7 and 20 years respectively. The last stress period is extended beyond the developmental phase to analyze the sustainability of the system. Table 22 shows the incremental development of the water resources.

Table 22: Simulation Stress Periods

#	Duration (yrs)	State	No. of active Reservoirs	No. of active Wells
1	1	Steady	0	0
2	4	Transient	6	12
3	7	Transient	10	28
4	20	Transient	13	50

3.12 Groundwater Simulations

Based on the construction of model and data analysis, simulations are performed that include modeling artificial recharge and pumping optimization. First the artificial groundwater recharge is modeled and then the wells are introduced. Four scenarios are developed for pumping in order to determine optimal location of the proposed 50 production wells. An optimization sensitivity exercise is performed by changing the position of the wells and adjusting the pumping rates in such a way that the daily consumption is maintained but the drawdown is minimized.

3.13 Model Testing

The MODLFOW model is tested against water table depths from the 6 available boreholes. Table 23 and Figure 16 illustrate the observed and simulated water table depths. In Table 23, percent change indicates that the difference between the observed and modeled values of water tables is low. Moreover, Figure 16 shows the simulated line almost overlying the observed representing accuracy of the model results in picking high and low depths for each borehole. The calibrated results are tested using statistics, summarized in section 3.13.1.

Table 23: Comparison of observed and simulated water table depths for model calibration.

Borehole	Observed	Simulated	Percent change
BH-1	49.6	46.8	5.6
BH-2	154.6	157.2	-1.7
BH-3	82	84.62	-3.2
BH-4	48	51.98	-8.3
BH-5	56	54.02	3.5
BH-6	81	77.93	3.8

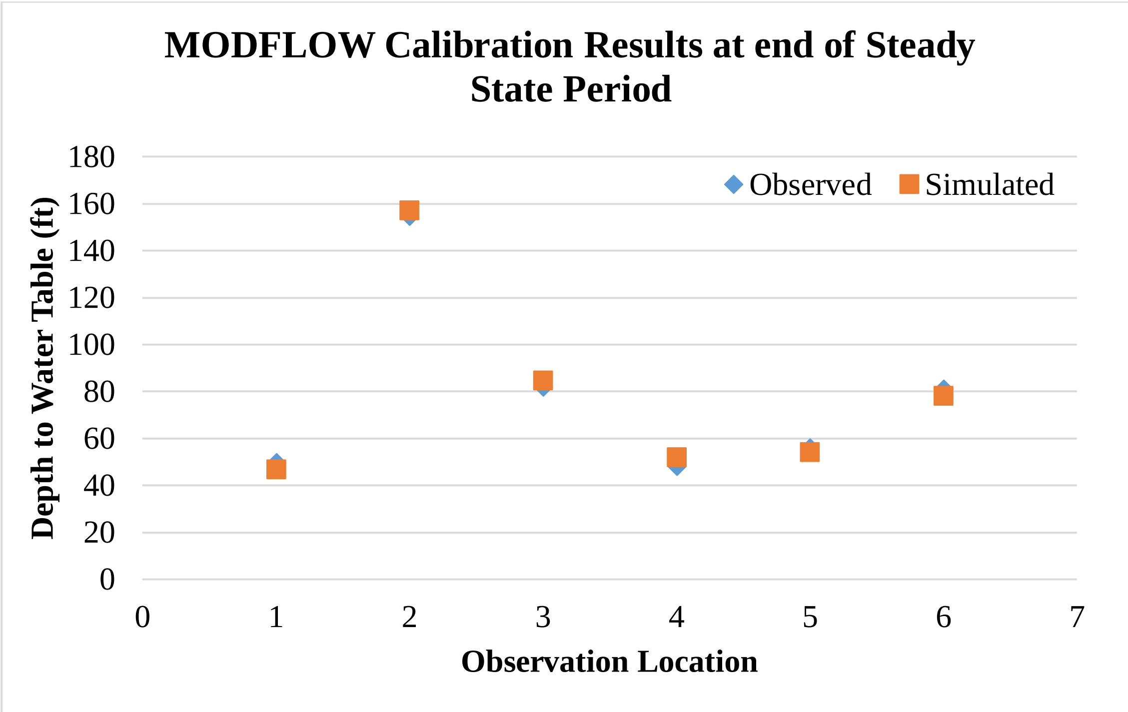

Figure 17: MODFLOW Calibration results

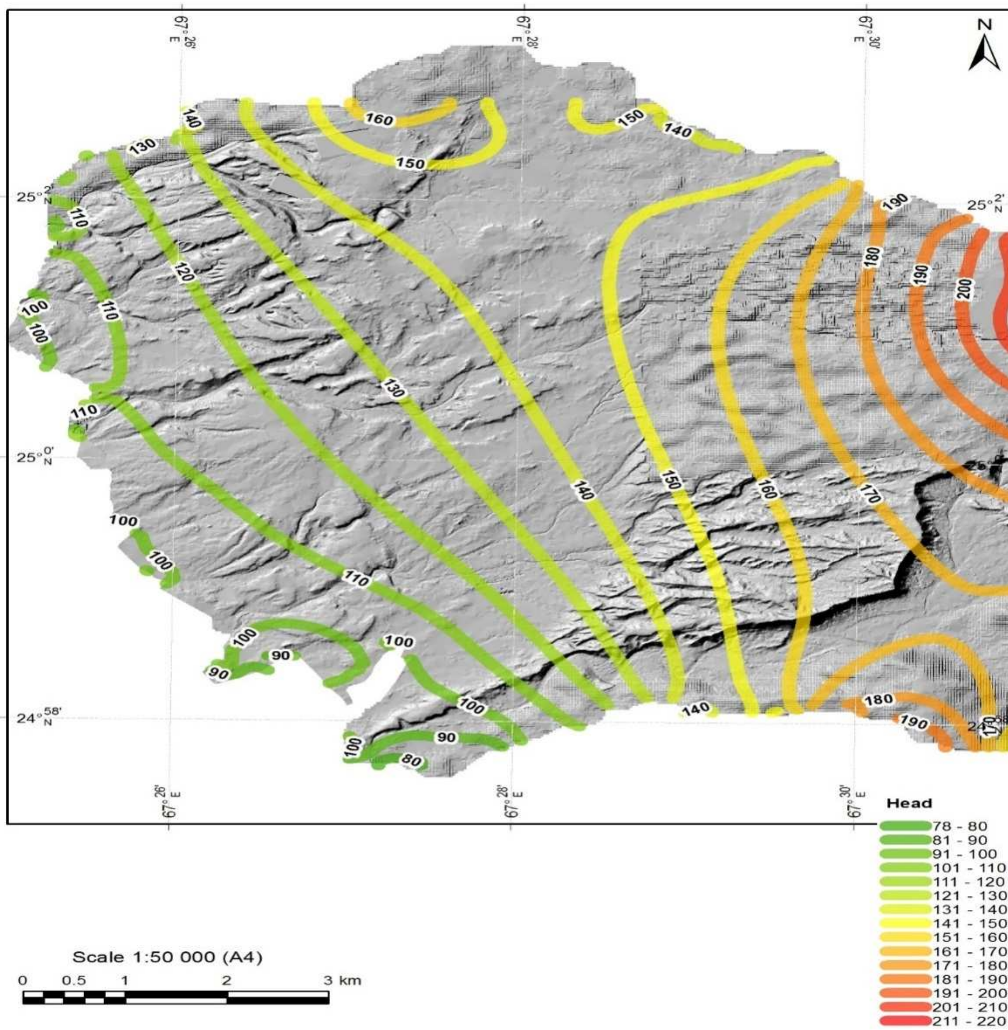

Figure 16a: Head Distribution after the steady state simulation

3.13.1 Statistical Tests on Calibration

The results of calibration are tested using statistics: Coefficient of Determination (R^2) to determine the goodness of fit and Chi-Square test that determines the distribution difference between observed and simulated(Legates and McCabe, 1999).

Co-efficient of determination: The range of co-efficient of determination is from zero to one. The values closer to one are considered to represent best agreement. R^2 is computed by using the following equation,

$$R^2 = \left[\frac{\sum_{i=1}^{n} (O_i - \overline{O})(P_i - \overline{P})}{\left(\sum_{i=1}^{n} (O_i - \overline{O})^2 \right)^{0.5} \left(\sum_{i=1}^{n} (P_i - \overline{P})^2 \right)^{0.5}} \right]^2 \qquad (16)$$

where O_i is the observed stream flow at time step *i*

$\bar{O}$ is the average observed stream flow

P_i is the model simulated stream flow at time step *i*

$\overline{P}$ is the average model simulated stream flow at time step *i* simulated by the model.

For the calibration, R^2 is 0.993 which indicates 99.3% accuracy and goodness to fit between observed and simulated values.

Chi-Square Test:

The range of chi-square is also from zero to one with one showing good result. It is calculated using the following equation,

$$X^2(C, E) = \sum_{i=A}^{i=Z} \frac{(Ci - Ei)^2}{Ei}$$

where C_A is the count (not the probability) of variable A, and E_A is the expected count of variable A.

The results of chi-square test shows 0.977 which is closer to 1 indicating the two variables do not have much difference.The results of statistical tests verify reliability on the MODFLOW model.

3.14 Results and Discussion; Groundwater modeling

3.14.1 Artificial Groundwater Recharge

The above described model was run for a scenario where no lakes were placed but the water was being pumped out with the same rate from the 50 wells which are being planned by the engineers. It goes on to show that in 12 years' time the groundwater would be depleted to such an extent that pumping will involve huge costs with draw downs going as low as 40 meters in some areas. Figure 17 gives contour map of drawdown distribution in the scenario where no recharge is being done.

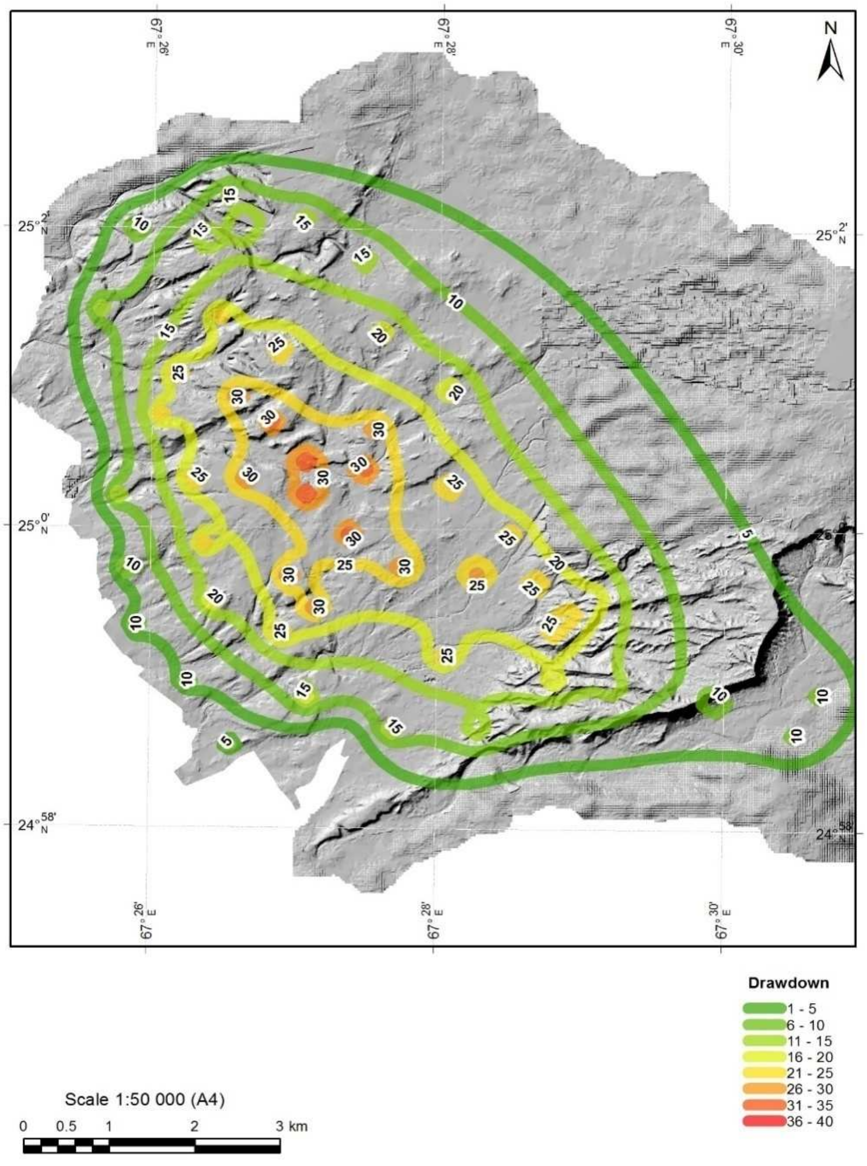

Figure 18: Drawdown contour map in the no-recharge scenario.

Another scenario was modelled wherein all the reservoirs were made active as per the original plan; however, no pumping was done just to get an estimate of the water that is being recharged, on which pumping shall be based. Table 24 shows the volume of cumulative water recharged and the volume that is being recharged per day.

Table 24: Artificial Recharge Volumes through the Simulation

#	Duration (yrs)	State	No. of active Reservoirs	Cumulative Volume Recharged (MG)	Daily Recharge (MGD)
1	1	Steady	0	0	0
2	4	Transient	6	7338.1	3.6
3	7	Transient	10	24370.9	5.6
4	20	Transient	13	83896.9	7.9

The volumes above give an idea of how pumping rates that can be easily assigned in order to meet the demand in a sustainable manner. A complete mass balance summary at the end of every time step is attached as Appendix E. KWSB gazette of 2015, which is attached as Appendix G, shows that in Karachi, the Bulk Water Supply rate for such housing schemes is 1.3$ per 1000 gallons for domestic and 2.2$ per 1000 gallons for commercial use. As explained above, the land-use with respect to water consumption can be divided equally in domestic and commercial use, the water saved from this recharge will be assessed at 1.75$ per 1000 gallons or 1,750$ per MG.

3.14.2 Pumping Out the Artificially Recharged Groundwater (Scenario 1)

As a general target, 5 MGD was planned to be taken out of the groundwater system to lower the burden on the municipal supply but owing to the recharge result we can afford to go a bit higher, as a result the pumping load was divided equally on all 50 wells randomly spread out throughout the DCK area to check the response of the system, henceforth known as Scenario 1. The results shall later help in optimizing the position of the wells and the pumping rates.

This scenario yielded a sustainable extraction of water from the aquifers, a summary of the simulation results is depicted in Table 25 and a comparison is done in Figure 18:

Table 25: Results of Simulation of Scenario 1

Duration (yrs)	State	No. of active Wells	No. of active Reservoirs	Cumulative Volume Recharged (MG)	Daily Recharge (MGD)	Cumulative Volume Extracted (MG)	Daily Pumping (MGD)
1	Steady	0	0	0	0	0	0
4	Transient	12	6	7338.1	3.6	2043.8	1.4
7	Transient	28	10	24370.9	5.6	10985.3	3.5
20	Transient	50	13	83896.9	7.9	59525.1	6.6

The wells were mostly placed looking at the population density and empty spaces for installation of well pumps. Figure 19 shows the drawdown contour map where it can be seen that draw downs up to 19m have been recorded in close proximity of the wells and lakes. Negative draw downs are due to recharge. The head distribution map of the scenario is attached as Appendix H.

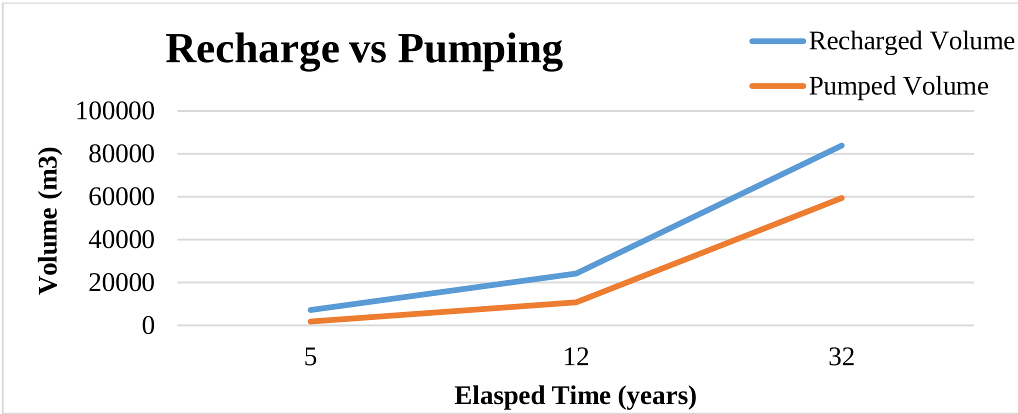

Figure 19: A graphical comparison of recharge vs pumping

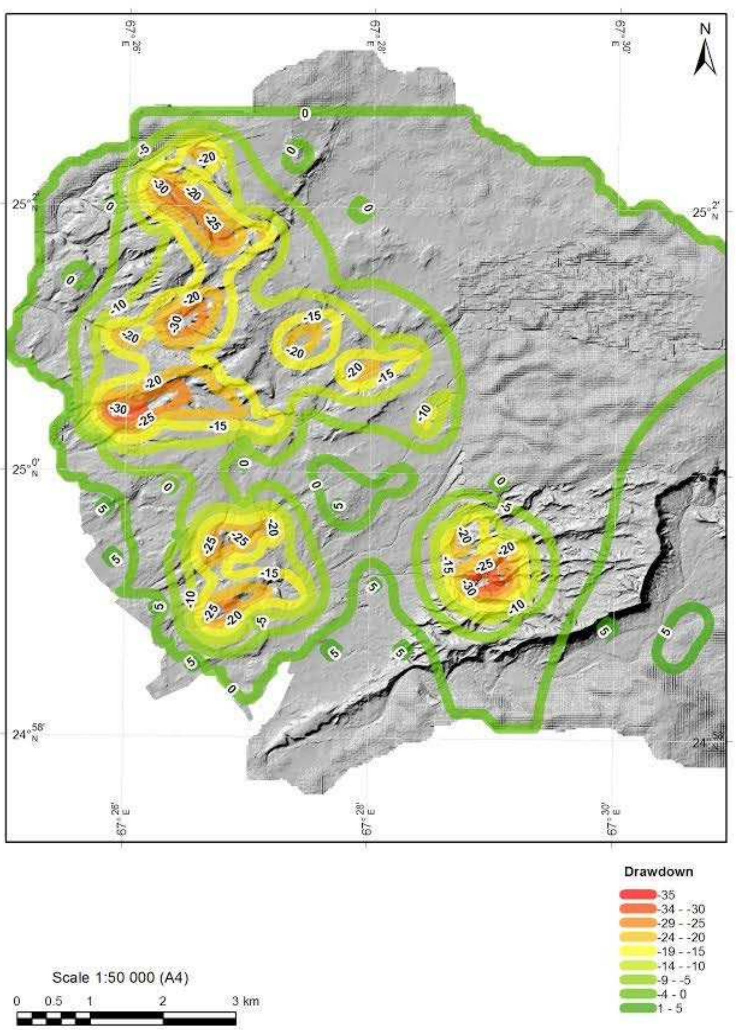

Figure 20: Drawdown Contour Map of Scenario 1

3.14.3 Well Placement Optimization (Scenario 2 and 3)

In order to restrict the drawdown yet keeping the same output in terms of volume, the position of the wells were strategically optimized. In Scenario 1, it can be seen that most of the recharge is either immediately downstream or in close proximity with the lakes, therefore, a Scenario 2 is developed with the wells placed around the lakes. This scenario has lessened the drawdown considerably; however, some spots continue to be under stress with draw downs up to 10m in some spots. The head distribution map of the scenario is attached as Appendix H. This arrangement of the wells will also increase the transmission cost of water to the end users as they are mostly concentrated in certain areas. Figure 20shows the drawdown contour distribution of Scenario 2.

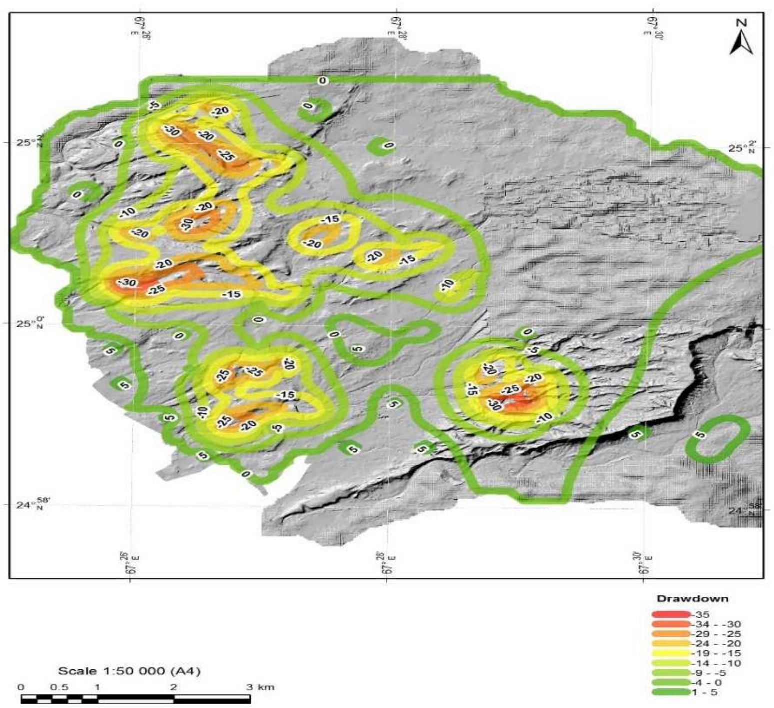

Figure 21: Drawdown Contour map of scenario 2

Observing the stress in a few areas on the contour map, a third scenario was developed to reduce the drawdown further. The wells were spaced out a little and placed in areas of higher recharge in order to incurless stress on the aquifer. This arrangement limits the drawdown of the system to a maximum of 5m in the vicinity of the pumping wells. The head distribution map of the scenario is attached as Appendix H. This arrangement seems to be a perfect placement for drawdown restriction, in order to further limit the drawdown, the pumping rates need to be optimized. Figure 21 shows the drawdown contour map for Scenario 3.

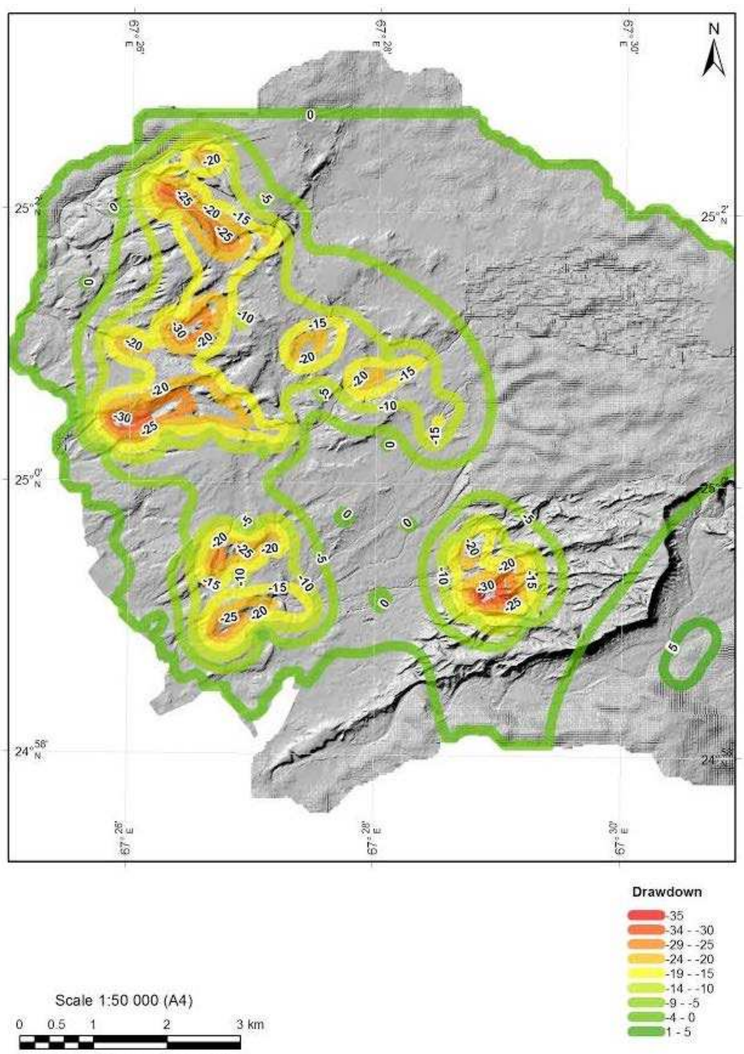

Figure 22: Drawdown Contour map of scenario 3

3.14.4 Pumping Rate Optimization:

The arrangement of wells in Scenario 3 seems to have controlled the drawdown as far as position of the wells with respect to recharge is concerned, however, in order to fully optimize the well system efficiency and bring the drawdown to a negligible minimum, the pumping rates need to be optimized in such a way that the total pumped out volume from the whole system remains the same. Therefore in the fourth scenario, pumping rates were lowered in the south eastern and mid-western wells whereas they were equally increased to balance out the total pumped volume in the North West and Western wells considering the recharge affect in the vicinity of the wells. The results limit the drawdown in the region to around 2m. It needs to be considered that the drawdown shown is with reference to the original state where the water table was 21.75m below the top elevation, therefore the pumping depth which was receding to around 40m in Scenario 1 has been brought down to around 23m in Scenario 4. Figure 22is the contour map for drawdowns of Scenario 4 and Figure 22a is the comparison of all average drawdown scenarios. The head distribution map of the scenario is attached as Appendix H.

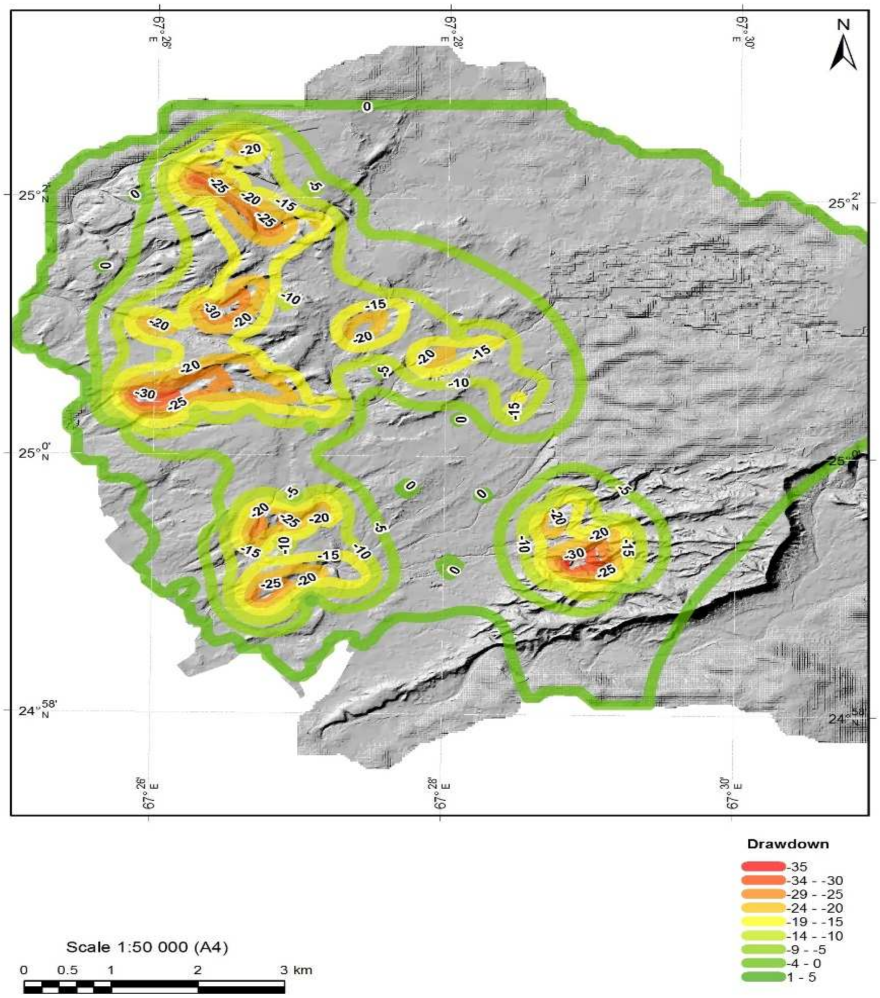

Figure 23: Drawdown Contour map of scenario 4

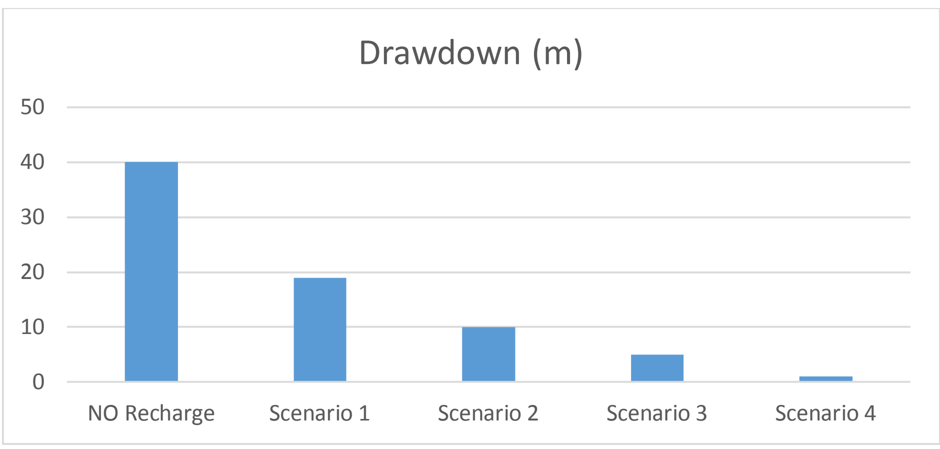

Figure 22a: Comparison of average drawdown in the scenarios

3.14.5 Cost-Benefit Analysis

As mentioned in section 2.4.1, the cost of bulk-water supply in Karachi for mixed used was assessed at 1,750$ per million gallons. Cost of one dam (9 meters high) was worked out by the project design consultant was 1.6 million USD. Multiplying the rate at the same cost for 13 dams (since more or less the size of the dams are the same), the total cost of the project (less the turbines and power house) comes out to be 20.8 million USD. The water that is being recycled is around 6.6 MGD, if the same volume was being purchased from KWSB, it would cost 11,550$ per day. At this rate, the project will pay back in 5 years ignoring the O&M costs of the dams. The cost that is being saved everyday can be further utilized to develop more water resources for DCK.

The cost that is being saved by limiting the drawdown as it has been brought down from 20m to 2m is also commendable and contributes to the economic benefit of the project. Some studies report that the cost of pumping increases by 6$ in a 24-hour operation if the water table drawsdown by a foot, which makes it approximately 18$ per meter. Assuming that we are saving 18$ per meter per day. We are saving tentatively $16,200 per day.

CHAPTER 4: SUMMARY CONCLUSIONS

This study aims to provide sustainable solutions towards the infrastructure development project for DHA City, Karachi, to serve a population of 1 million. The possible investigations from this research work included potential of hydroelectric power production from small artificial lakes in the study area and subsequent artificial groundwater recharge potential and succeeding adequate pumping. The extensive work for groundwater estimations using MODFLOW and ANN and MPC for hydroelectricity has been done to provide the impact analysis, following cost-benefit analysis for each.

4.1 Summary and Conclusions

For hydroelectric model simulation, the controllers are determined to be useful tool for control dam operations. The results show 13.92 MW of green energy production from 13 possible dams that will collectively save daily amount of approximately $60,000. The initial investment in dam construction and O & Mis estimated to recover by the end of year 2 of commissioning which draws attention toward the long-term benefit for the new city, economically and environmentally.

Groundwater quantity modeling having greater focus of this study delivers elucidating results where, in 20 years of time, recharge is approximately 8 MGD and pumping is 6.6 MGD. This interprets steady and sustainable conditions of groundwater and consequent utilization for multiple municipal purposes. In addition, cost-benefit analysis for groundwater concludes that water cost (cost of dams) will pay back in 5 years ignoring the O & M costs of dams. The saving of every day water cost can be utilized for future water resources development for the city.

4.2 Shortcomings

Although the study comprises of diverse results, there exist shortcomings, majorly including paucity of data (e.g. borelog dataset) which on availability would have produced detailed and more accurate results. This study is recharge under natural head; howsoever, further investigation can produce some engineering inventions viable for long-term. As such, results are preliminary in regards to an actual artificial recharge design, and thus should be treated with caution.

Furthermore, this study is limited to water quantity analysis. Inclusion of water quality will significantly hamper the mentioned cost-benefit analysis, for both hydroelectricity and groundwater. One of the examples is treatment cost for improvement in water quality.

A disclaimer of sorts also needs to be issued here to the readers that all the costs stated in the study may vary from region to region and may also include a lot of overheads which were beyond the scope of the study.

4.3 Future Research

The initiative for a sustainable water resources system is one of the most needed infrastructural component in town planning for a semi-arid region. Artificial lakes and consequent hydropower generation and groundwater extraction seems a good way of reutilizing treated wastewater. The cost benefit analysis provides numbers in support. If implemented, detailed feasibility can support the significance and impact of the proposed idea.

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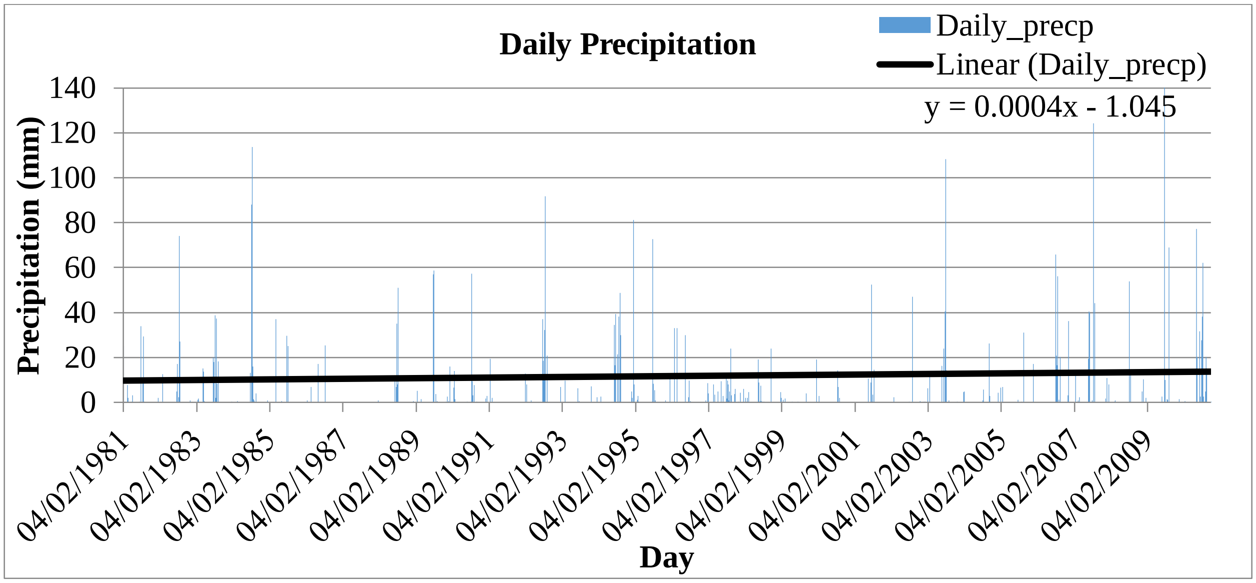
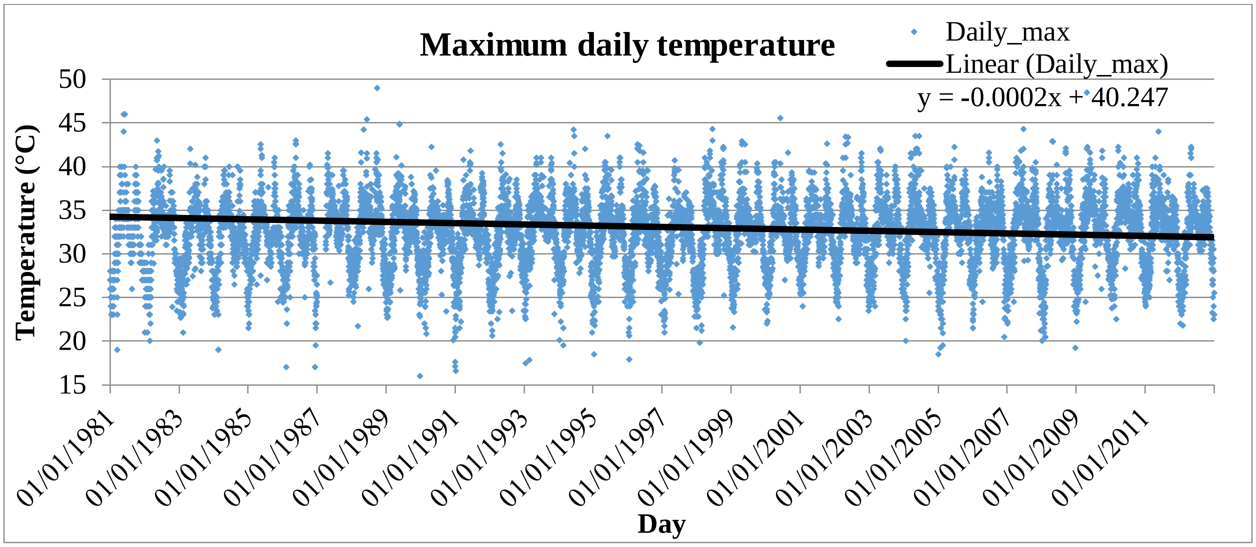
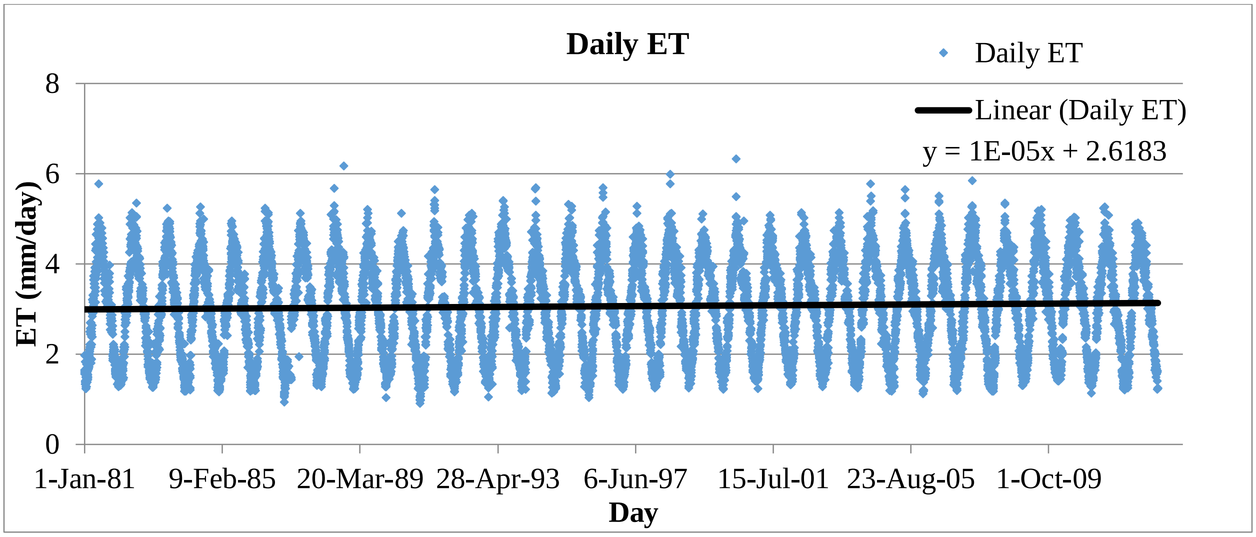
APPENDICES

APPENDIX – A   Rainfall Analysis



APPENDIX – B Evapotranspiration Analysis





APPENDIX – C Ekistics Planning

Figure C1: Land use categories of DHA city.

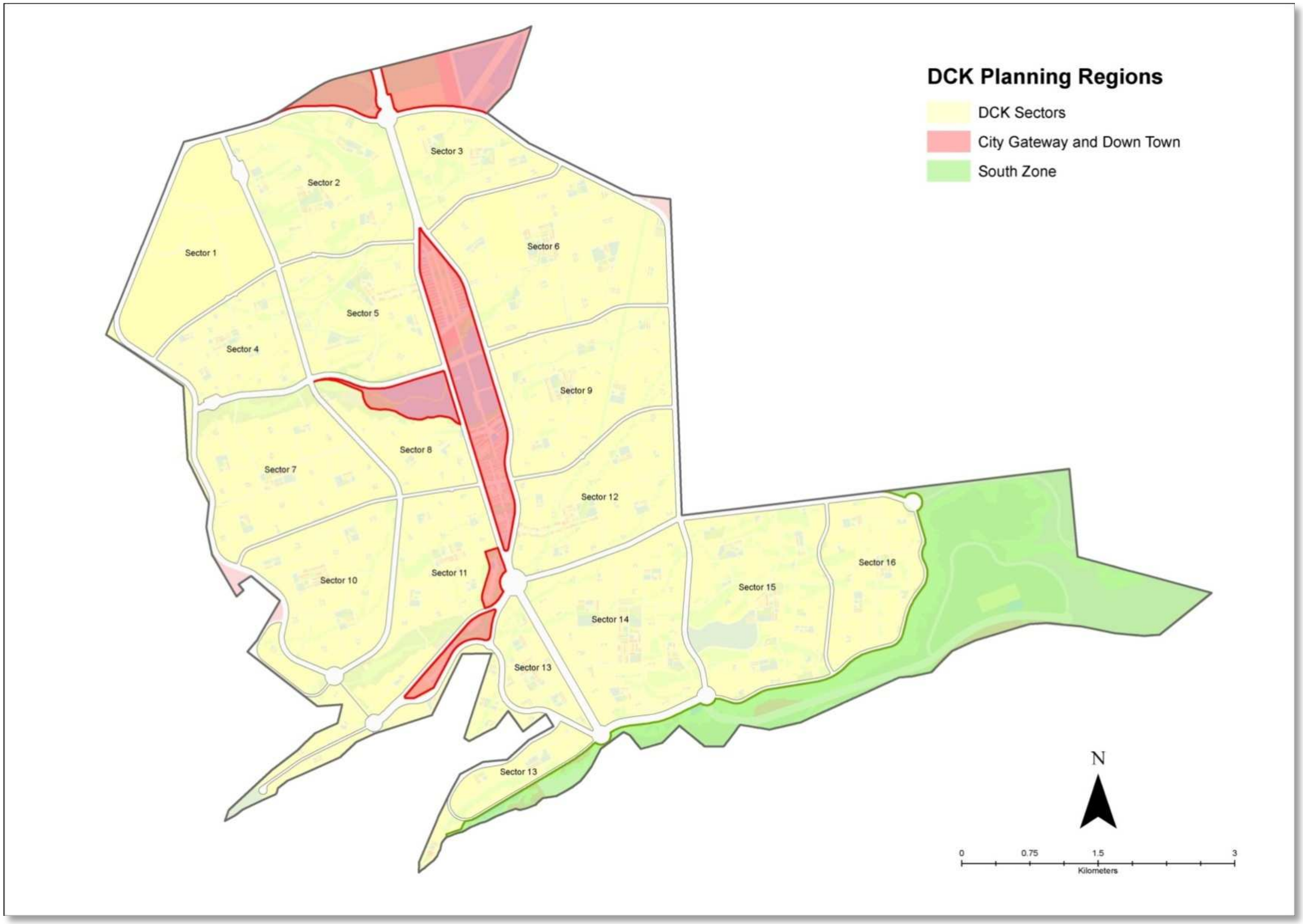


Figure C2: DCK Planning Regions

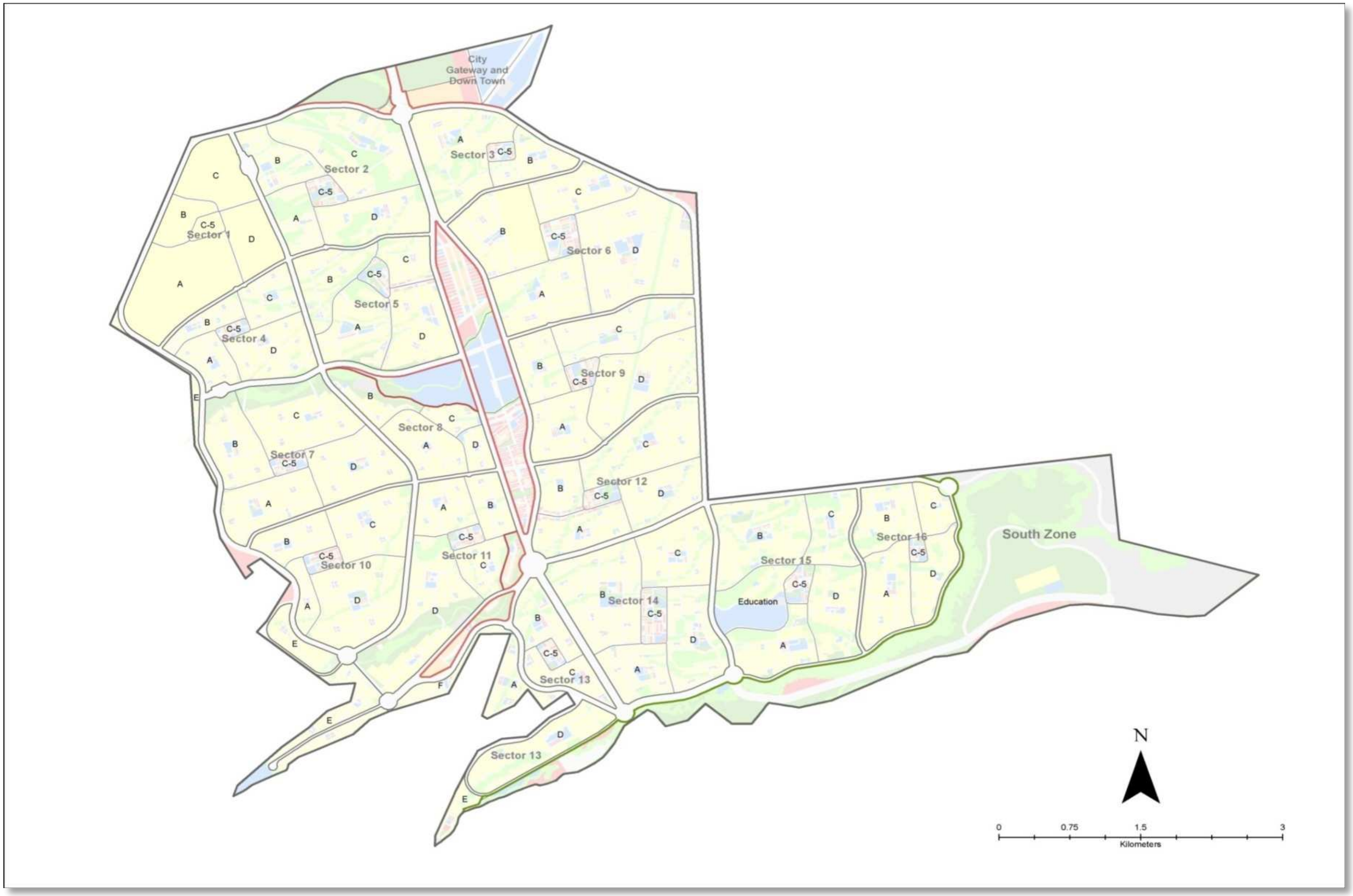
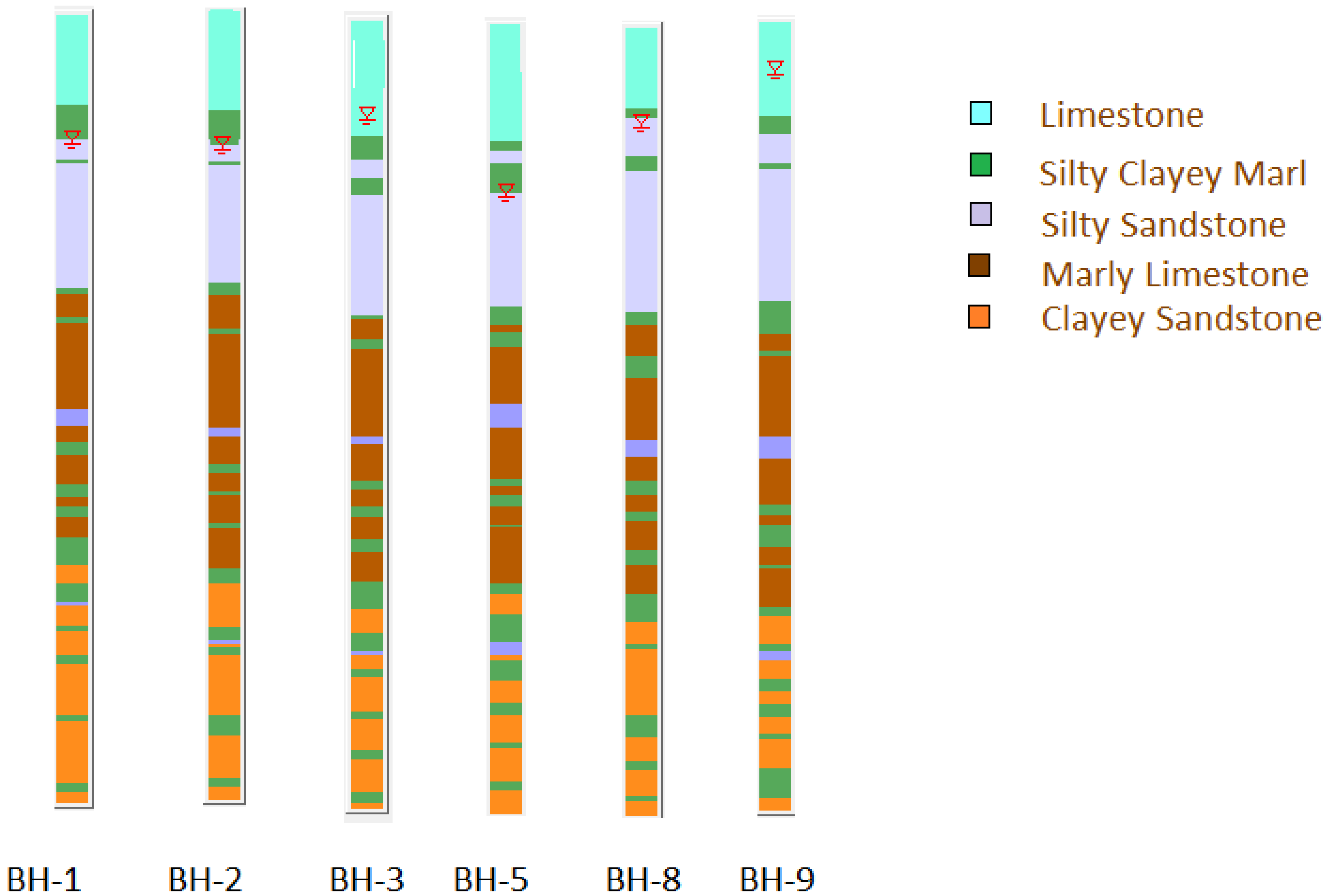


Figure C3: DCK Sector Planning on Ekistic Model

APPENDIX – D Borelog Data



APPENDIX - E Mass Balance Summary

```
 DRAWDOWN WILL BE SAVED ON UNIT   38 AT END OF TIME STEP  487, STRESS PERIOD    4
1
  VOLUMETRIC BUDGET FOR ENTIRE MODEL AT END OF TIME STEP  487, STRESS PERIOD   4
  ------------------------------------------------------------------------------

     CUMULATIVE VOLUMES      L**3       RATES FOR THIS TIME STEP      L**3/T
     ------------------                 ------------------------

           IN:                                      IN:
           ---                                      ---
             STORAGE =     3548100.5000               STORAGE =         11.1152
       CONSTANT HEAD =   413666272.0000         CONSTANT HEAD =      35087.3281
               WELLS =           0.0000                 WELLS =          0.0000
     SPECIFIED FLOWS =           0.0000       SPECIFIED FLOWS =          0.0000
     RESERV. LEAKAGE =   317584544.0000       RESERV. LEAKAGE =      29765.9199

            TOTAL IN =   734798912.0000              TOTAL IN =      64864.3633

          OUT:                                     OUT:
          ----                                     ----
             STORAGE =    55627192.0000               STORAGE =        364.4902
       CONSTANT HEAD =   453694592.0000         CONSTANT HEAD =      39516.8125
               WELLS =   225465856.0000                 WELLS =      24981.0000
     SPECIFIED FLOWS =           0.0000       SPECIFIED FLOWS =          0.0000
     RESERV. LEAKAGE =           0.0000       RESERV. LEAKAGE =          0.0000

           TOTAL OUT =   734787648.0000             TOTAL OUT =      64862.3008

            IN - OUT =       11264.0000              IN - OUT =          2.0625

 PERCENT DISCREPANCY =           0.00     PERCENT DISCREPANCY =          0.00




        TIME SUMMARY AT END OF TIME STEP  487 IN STRESS PERIOD    4
                    SECONDS     MINUTES      HOURS       DAYS        YEARS
                    -----------------------------------------------------------
   TIME STEP LENGTH 1.29511E+06  21585.       359.75      14.990     4.10396E-02
 STRESS PERIOD TIME 6.30721E+08 1.05120E+07 1.75200E+05  7300.0      19.986
         TOTAL TIME 1.00914E+09 1.68190E+07 2.80317E+05  11680.      31.978
1
```

APPENDIX – F KE Tariff

REVISED SCHEDULE OF ELECTRICITY TARIFF FOR K-ELECTRIC LIMITED (Formerly KESC)

Notified vide SRO # 677 dated: 14-Jul-2015

A-1 - GENERAL SUPPLY TARIFF - RESIDENTIAL

Sr. No.	TARIFF CATEGORY / PARTICULARS	Fixed Charges Rs/kW/M	Variable Charges Rs / kWh	
a)	For Sanctioned load less than 5 kW			
i	Upto 50 Units		2.00	
	For Consumption exceeding 50 Units			
ii	1 - 100 Units		5.79	
iii	101 - 200 Units		8.11	
iv	201 - 300 Units		10.20	
v	301 - 700 Units		16.00	
vi	Above 700 Units		18.00	
b)	For Sanctioned load 5 kW & above		Peak	Off- Peak
	Time of Use		18.00	12.50

As per Authority's decision Residential Consumers will be given the benefit of only one previous slab
Consumption exceeding 50 units but not exceeding 100 units will charged under the 1-100 slab.
Under tariff A-1, there shall be minimum monthly customer charge at the following rates:
a) Single Phase Connections: Rs. 75/- per consumer per month
b) Three Phase Connections: Rs. 150/- per consumer per month

A-2 - GENERAL SUPPLY TARIFF - COMMERCIAL

Sr. No.	TARIFF CATEGORY / PARTICULARS	Fixed Charges Rs/kW/M	Variable Charges Rs / kWh	
a)	For Sanctioned load less than 5 kW		18.00	
b)	For Sanctioned load 5 kW & above	400	16.00	
			Peak	Off- Peak
c)	Time of Use	400	18.00	12.50

Under tariff A-1, there shall be minimum monthly customer charge at the following :
a) Single Phase Connections: Rs. 175/- per consumer per month
b) Three Phase Connections: Rs. 350/- per consumer per month

B - INDUSTRIAL SUPPLY TARIFFS

Sr. No.	TARIFF CATEGORY / PARTICULARS	Fixed Charges Rs/kW/M	Variable Charges Rs / kWh	
B1	Less than 5 kW (at 400 / 230 Volts)		14.50	
B2(a)	5-500 kW (at 400 Volts)	400	14.00	
B3(a)	For all loads upto 5000 KW (at 11, 33 kV)	380	16.14	
B4(a)	For all loads upto 5000 KW (at 66, 132 kV)	360	15.74	
	Time of Use		Peak	Off- Peak
B2(b)	5-500 kW (at 400 Volts)	400	18.00	12.29
B3(b)	For all loads upto 5000 KW (at 11, 33 kV)	380	18.00	12.20
B4(b)	For all loads upto 5000 KW (at 66, 132 kV)	360	18.00	12.10
B5	For all loads (at 220 kV & above)	340	18.00	12.00

For B1 consumers there shall be a fixed minimum charge of Rs. 350 per month.
For B2 consumers there shall be a fixed minimum charge of Rs. 2,000 per month.
For B3 consumers there shall be a fixed minimum charge of Rs. 50,000 per month.
For B4 consumers there shall be a fixed minimum charge of Rs. 500,000 per month.
For B5 consumers there shall be a fixed minimum charge of Rs. 1000,000 per month.

C - SINGLE-POINT SUPPLY FOR PURCHASE IN BULK BY A DISTRIBUTION LICENSEE AND MIXED LOAD CONSUMERS NOT FALLING IN ANY OTHER CONSUMER CLASS

Sr. No.	TARIFF CATEGORY / PARTICULARS	Fixed Charges Rs/kW/M	Variable Charges Rs / kWh	
C1	For supply at 400 / 230 Volts			
a)	Sanctioned load less than 5 kW		15.00	
b)	Sanctioned load 5 kW & upto 500 kW	400	14.50	
C2 (a)	For supply at 11, 33 kV upto and including 5000 kW	380	14.30	
C3 (a)	For supply at 132 and above, upto and including 5000 kW	360	14.20	
	Time of Use		Peak	Off- Peak
C1 (c)	For supply at 400 / 230 Volts 5 kW & upto 500 kW	400	18.00	12.50
C2 (b)	For supply at 11, 33 kV upto and including 5000 kW	380	18.00	12.30
C3 (b)	For supply at 132 kV upto and including 5000 kW	360	18.00	12.20

D - AGRICULTURE TARIFF

Sr. No.	TARIFF CATEGORY / PARTICULARS	Fixed Charges Rs/kW/M	Variable Charges Rs / kWh	
D1	For all loads		11.50	
	Time of Use		Peak	Off- Peak
D2	For all loads	200	10.35	8.85

E - TEMPORARY SUPPLY TARIFFS

Sr. No.	TARIFF CATEGORY / PARTICULARS	Fixed Charges Rs/kW/M	Variable Charges Rs / kWh
E1 (i)	Residential Supply		21.34
E1 (ii)	Commercial Supply		24.24
E2 (i)	Industrial Supply		20.94
E2 (ii)	Bulk Supply		
	(a) at 400 Volts		21.34
	(b) at 11 kV		20.94

For the categories of E1 (i&ii) and E2 (i&ii) above, the minimum bill of the consumers shall be Rs. 50/- per day subject to a minimum of Rs. 500/- for the entire period of supply, even if no energy is consumed.

G - PUBLIC LIGHTING

Sr. No.	TARIFF CATEGORY / PARTICULARS	Fixed Charges Rs/kW/M	Variable Charges Rs / kWh
	Street Lighting		15.00

Under Tariff G, there shall be a minimum monthly charge of Rs. 500/- per month per kW of lamp capacity

APPENDIX –G KWSB Tariff

3) Flats

DOMESTIC/UN-METERED (IN SQUARE FEET)	Proposed Tariff WATER CHARGES (PER MONTH)
Up to 500	110
501 to 800	166
801 to 1000	196
1001 to 1200	296
1201 to 1500	449
1501 to 1800	786
1801 to 2000	997
2001 to 2500	1256
2501 to 3000	1529
3001 to 3500	1833
3501 to 4000	2157
4001 to 5000	3153
Above 5000	4056

4) Miscellaneous

(i)	Offices	Same as Domestic Tariff of Flats depending upon covered area
(ii)	Shops	Rs. 52/- Per Month
(iii)	Dhobi Ghat, Restaurants, Agriculture, Nurseries, Block Thallas, Cattle Ponds, Hammams	Rs. 222/- per 1000 Gallons or Double as Domestic Tariff of Plot depending upon covered area
(iv)	Commercial High Rise Buildings and Hotels (Single Unit)	Rs. 222/- per 1000 Gallons or Double as Domestic Tariff of Plot depending upon covered area
(v)	Colleges, Schools, Clinics and Hospitals	Rs 222/- per 1000 Gallons or Double as Domestic Tariff of Plot depending upon covered area
(vi)	Marriage Halls, Lawns and Clubs	Double of Commercial Tariff

5) Bulk Supply

(i)	Domestic	Rs. 130/- per 1000 Gallons
(ii)	Industrial	Rs. 222/- per 1000 Gallons
(iii)	Commercial	Rs. 222/- per 1000 Gallons
(iv)	Hydrant	Rs. 222/- per 1000 Gallons

(MUHAMMAD ASLAM KHAN)
Deputy Managing Director (RRG)
Karachi Water & Sewerage Board

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PART – I-A

KARACHI WATER & SEWERAGE BOARD

the Deputy Managing Director (RRG)

No. KW&SB/DMD/RRG/2015/ 145 Date: 16-09-2015

NOTIFICATION

No. MD/KW&SB/2015/1768/L dated 17-08-2015, pursuant to the powers conferred upon KW&SB by the Government of Sindh under Clause-V of Para-4, Government Gazette Notification published on October 4th, 2001 to revise the tariff @ 9% annually under Sub-Section-4 of Section (8) of the Karachi Water & Sewerage Board Act-1996, in respect of Revised Schedule for Water Service Charges in different Categories as approved by the Chairman, KW&SB., The Managing Director, KW&SB hereby gives effect on and from 1st July-2015 and notify Revised Rate Schedule (amended).

1) Plots (Residential)

DOMESTIC/UN-METERED (IN SQUARE YARD)	Proposed Tariff WATER CHARGES (PER MONTH)
Up to 60	106
60 to 120	144
121 to 200	222
201 to 300	328
301 to 400	459
401 to 600	675
601 to 1000	976
1001 to 1500	2006
1501 to 2000	2606
2001 to 2500	3321
2501 to 3000	4212
3001 to 3500	5132
3501 to 4000	6106
4001 to 4500	7125
4501 to 5000	8406
Above 5000	9715

2) Additional Storey 50% of Ground Floor (for each storey)

APPENDIX –H Head Distribution Maps for all Scenarios

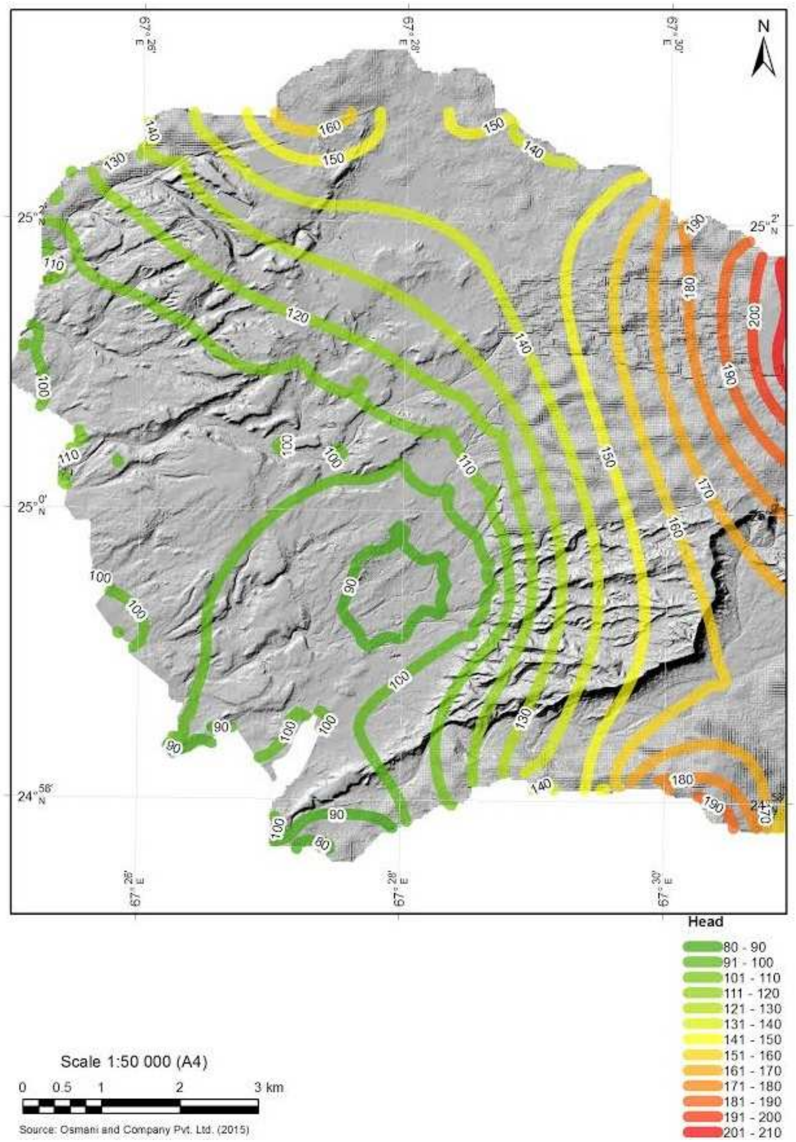

Figure F1: Head Distribution Map of No-Recharge Scenario

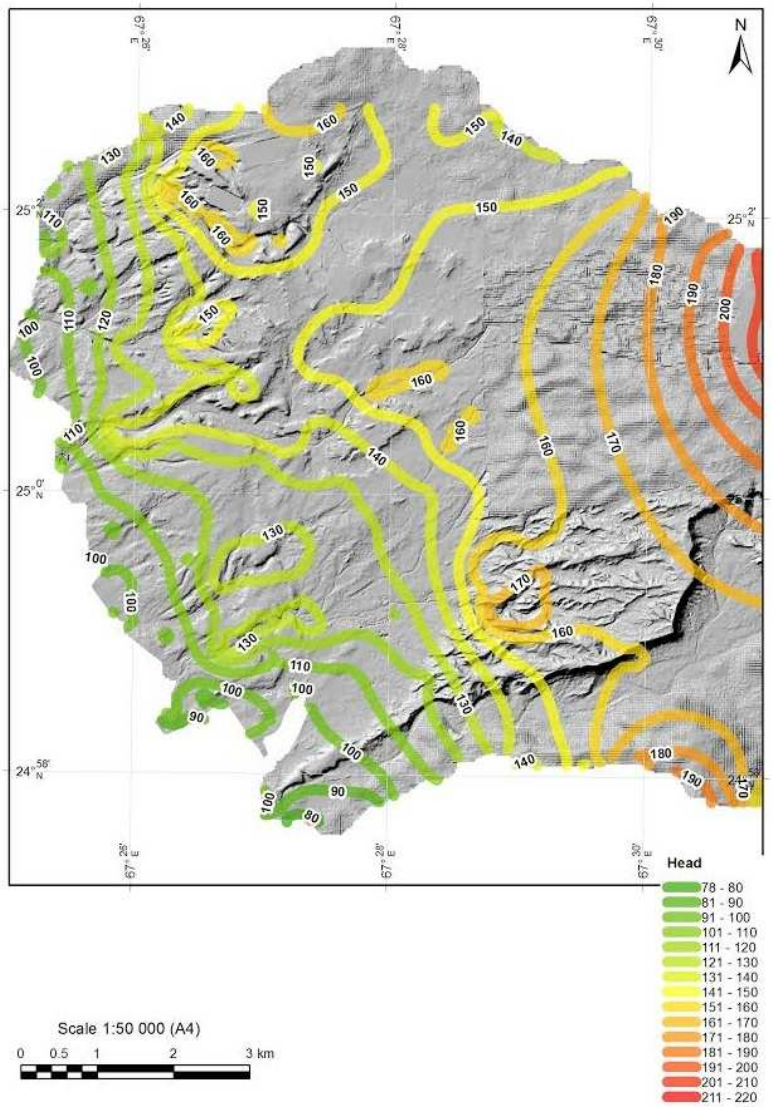

Figure F2: Head Distribution Map of Scenario 2

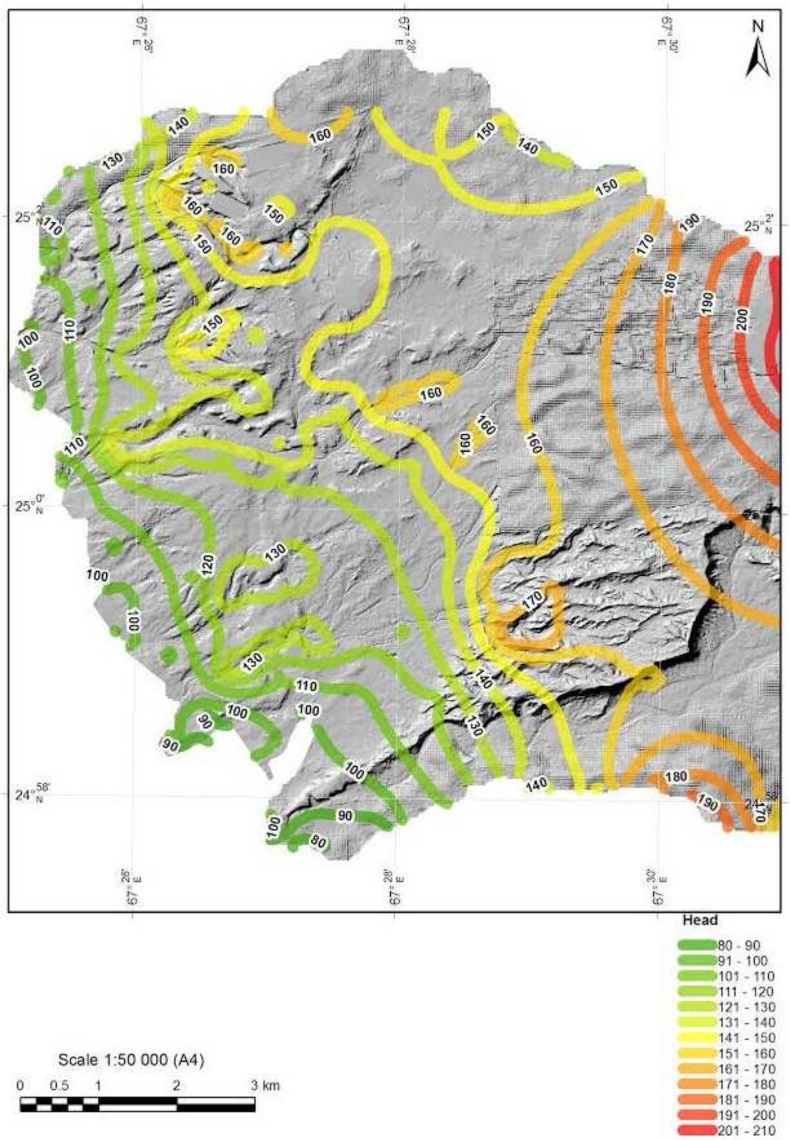

Figure F3: Head Distribution Map of Scenario 3

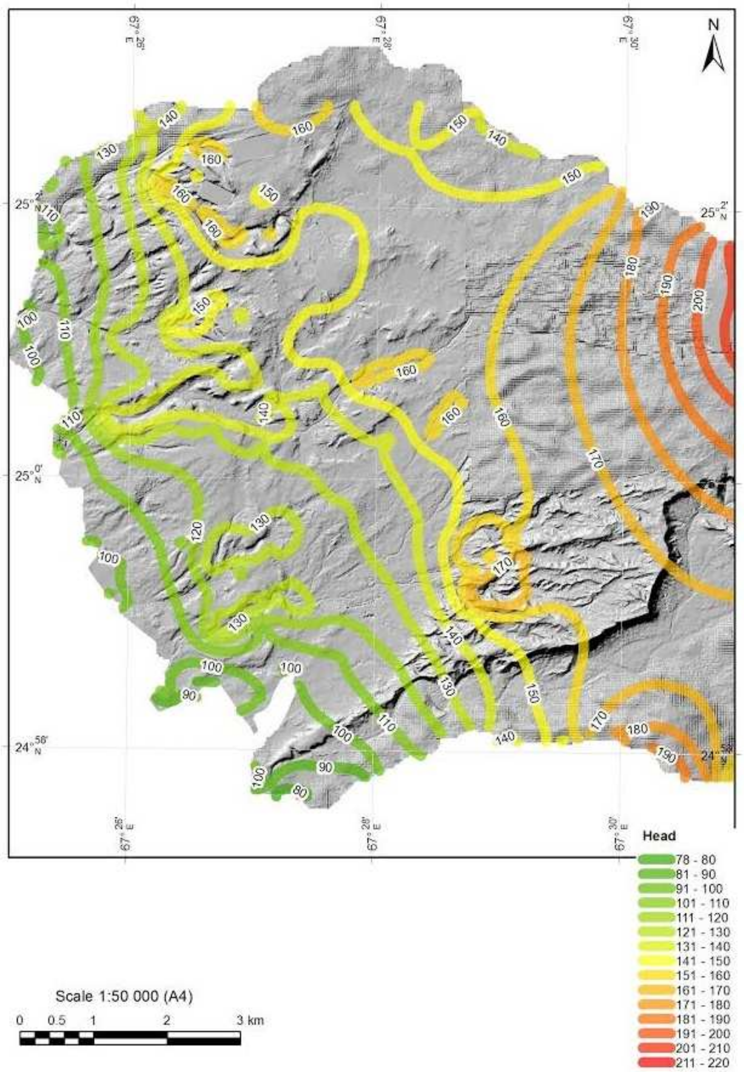

Figure F4: Head Distribution Map of Scenario 4