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para a Ricarda

Summary

The implementation of water distribution networks for irrigation represents an important investment for the development of many countries and for food security. These investments are mostly executed based solely on engineering designs, resting on the minimization of the network's costs, without regard to the balance between the life-cycle irrigation benefits, life-cycle costs of the investment and the hydraulic conditions. Such designs and investments can, in the long-term, incur unsustainable operation and maintenance costs as well as poor cost recovery.

The imperative motivation of this work was the necessity for bridging engineering, agricultural and economic reasoning in dimensioning water distribution networks, i.e. the necessity for inclusive life-cycle cost-benefit optimization approaches, for the simultaneous dimensioning of the network and the appraisal of the investment.

For this purpose, a cost-benefit framework for project appraisal was developed as an optimization model that delivers the net-present value of the investment and optimal design of the network for selected rates of return, i.e. size of the network (or number of hydrants in the optimal solution), pipeline diameters, pump system capacity (discharges, heads and power demanded per irrigation shift), as well as the optimal spatial distribution of simultaneous operating hydrants per irrigation shift.

The natural mathematical formulation of the problem is of the mixed integer non-linear type. In this work, a linear approximation with a mixed integer formulation was developed, based on piece-wise linear approximations of the non-linear hydraulic constraints and non-linear energy costs in the objective function of the model. The efficient approximations and modern solvers allowed the model results to be achieved in short computation times.

The model was applied to a case-study in Upper-Egypt, an agricultural settlement and irrigation water distribution network project included in Egypt's program for desert land reclamation. The model produces optimal network designs according to the project returns expected by the implementation agency, as well as expectation on diesel and food price

(benefits) developments. The results in the different scenarios showed that more irrigation hydrants (and agricultural area) will only be included in the optimal solution, if their marginal benefit contribution to the objective function is at least as large as their marginal costs. In this process, the model accounts for the different elevations of hydrants, i.e. the benefits inherent to each hydrant are equated to the hydrant's cost of connection and operation at the different elevations and distances to the pump system (head losses). This is a major result for implementing agencies which need to decide on the best network size, given topographical characteristics. The model is also able to optimally determine the shift pattern simultaneously with the size of the network. It derives a shift pattern that best balances the trade-offs between energy and initial investment costs for the given scenario and the project's demanded level of return. Including cost recovery concerns showed a strong economic non-sustainability for the case-study. Charging farmers only for energy costs would result in very low net-returns to a family's land, work and management, and would probably not allow family subsistence.

The model has proved to be a major tool for project appraisal, allowing a full economic estimation of the social costs and benefits involved in the implementation of water distribution networks for irrigation. The agency can compare projects in different locations, or decide between implementing larger or smaller irrigation settlements. Decision criterion can be based on the project's rates of return and also complemented with other development objectives like employment. These complementary objectives can be used to help decide between projects with equal rates of return. The present work shows that investments in water distribution networks for irrigation can and should be made using much more comprehensive engineering, agricultural and economic methods.

Zusammenfassung

Die Implementierung von Wasserverteilungsnetzen bei der landwirtschaftlichen Bewässerung ist für viele Länder des globalen Südens eine sehr wichtige Investition im Hinblick auf Entwicklung und Nahrungssicherheit der eigenen Bevölkerung. Als Entscheidungsgrundlage für Investitionen dienten jedoch überwiegend nur Analysen der Kostenminimierung unter Berücksichtigung von hydraulischen Bedingungen. Investitionsanalysen, die neben den hydraulischen Bedingungen und der Lebenszykluskosten auch Aspekte des Lebenszyklusnutzens mit einbeziehen, finden nicht statt. Solche einseitigen Investitionsanalysen und die daraus resultierenden Entscheidungen für konkrete Designs führen langfristig zu nicht nachhaltigen Operations- und Erhaltungskosten und zu einer entsprechend problematischen Kostendeckung. Motivation für die vorliegende Arbeit ist die Erkenntnis, dass für die beschriebene Problematik dringend ein interdisziplinärer Ansatz in der Modellierung von Verteilungsnetzen notwendig erscheint, der technisch-hydraulische Fakten mit einer fundierten Landwirtschaftsexpertise und mit ökonomischen Kosten-Nutzen-Ansätzen kombiniert. Dadurch soll ein neuer Kosten-Nutzen-Ansatz entstehen, der die simultane Evaluierung der Investition mit der Bestimmung der optimalen Größe und Dimensionierung des Netzwerkes erlaubt. Um dies zu erreichen, wurde ein Kosten-Nutzen-Ansatz entwickelt, basierend auf einem Optimierungsmodel, das den Kapitalwert der Investition für verschiedene erwartete Renditen maximiert. Die Lösung des Problems besteht aus der Bestimmung der optimalen Größe der Siedlung und des Verteilungsnetzes (Anzahl der Hydranten und räumliche Position), dem Durchmesser der Leitungen, der Größe der Pumpanlage (notwendiger Abfluss, Druck und Pumpenleistung) und auch der optimalen räumlichen Verteilung der operierenden Hydranten pro Bewässerungsschicht. Das Problem wird dargestellt in Form eines nichtlinearen, gemischt-ganzzahligen Programms. Für die Problemstellung wurde in der vorliegenden Arbeit ein Lösungsansatz in Form von einer linearen Approximation mittels eines linearen gemischt-ganzzahligen Models entwickelt. Dafür wurden die Nicht-Linearitäten in den hydraulischen Bedingungen und die

Energiekosten in der Zielfunktion durch die Konstruktion von abschnittsweise linearen Funktionen erfolgreich modelliert. Das Model wurde auf eine Fallstudie in Oberägypten angewandt, eine landwirtschaftliche Siedlung mit Verteilungsnetz für Bewässerung im Rahmen des ägyptischen Landgewinnungsprogramms. Für diesen Standort konnte das Model optimale Systeme ermitteln, entsprechend der Rendite-Erwartungen der Implementierungsbehörde, als auch entsprechend der Erwartungen bzgl. zukünftiger Entwicklungen von Diesel- und Nahrungsmittelpreisen. Im Analyseprozess des vorliegenden Models werden die Lage und die Höhe der Hydranten im Leitungsnetz berücksichtigt, d.h. der zusätzliche Nutzen wird den Kosten der Verbindung zum Netz und zur Operation dieses Hydranten gegenübergestellt. Dies ist ein besonders wichtiges Ergebnis für eine Implementierungsbehörde, die nun über eine Siedlung und die mögliche Größe des Systems auf der Basis einer soliden Kosten-Nutzen-Analyse entscheiden kann. Das Model bestimmt auch die optimalen Bewässerungsschichten simultan mit der Größe und der entsprechenden Dimensionierung des Leitungsnetzes. In den Schichten wird eine räumliche Verteilung der gemeinsam operieren den Hydranten bestimmt, welche die Trade-offs zwischen Energiekosten und Investitionen für die verschiedenen Szenarien und gewünschten Renditen ausbalanciert. Berechnungen zur Kostendeckung zeigen eine nicht nachhaltige Situation für die Investition in das Wasserverteilungssystem und die Pumpanlage der landwirtschaftlichen Siedlung. Eine Beteiligung der Landwirte ausschließlich an den Betriebs- und Erhaltungskosten der Gesamtanlage zeigt, dass den Landwirten ein sehr geringes Nettoeinkommen im Monat übrig bleibt. Dies stellt eine prekäre, nicht nachhaltige Situation der Investition dar.

Das Modell liefert eine umfassende ökonomische Analyse der sozialen Kosten und Nutzen einer Investition und macht Entscheidungsträgern diese Ergebnisse zugänglich. Implementierungsbehörden können mithilfe des Models Entscheidungen über optimale Größe und Dimensionierung der Gesamtanlage an einem bestimmten Standort treffen. Die Analyse ist außerdem erweiterbar und kann noch durch zusätzliche Entwicklungsziele, wie z.B. Beschäftigung, ergänzt werden.

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Acronyms

ANN Artificial Neural Networks

AHD Aswan High Dam

BIP Binary Integer Programming

BB Branch-and-Bound

CLM Castilla-La Mancha Irrigation Scheme

CO Colony Optimization

CE Cross Entropy

CPI Consumer Price Index

CBA Cost-Benefit Analysis

DP Dynamic Programming

EGP Egyptian Pounds

FC Field Capacity

FAO Food and Agriculture Organization of the United Nations

GAMS General Algebraic Modeling System

GA Genetic Algorithms

GDP Gross Domestic Product

GIS Geographical Information System

HS Harmony Search Algorithm

HW Hazen-Williams Equation

HGL Hydraulic Gradient Level

IP Integer Programming

IRR Internal Rate of Return

KLB Kalabsha Irrigation Scheme

LPG Linear Programming Gradient

LP Linear Programming

MAD Management Maximal Allowed Deficit

MA Memetic Algorithm

MALR Ministry of Agriculture and Land Reclamation

MILP Mixed Integer Linear Programming

MINLP Mixed Integer Non-linear Programming

NLPG Non-linear Programming Gradient

NLP Non-linear Programming

NPV Net present value

PVC Polyvinyl Chloride

OM Operation and Maintenance Costs

PSF HS Parameter-free Harmony Search

PV Present Value

RAW Readily Available Water

SS Scatter Search

SFLA Shuffled Frog Leaping Algorithm

SA Simulated Annealing

SOS1 Special Ordered Set of Type One

SOS2 Special Ordered Set of Type Two

TS Tabu Search

TAW Total Available Water in the Root Zone of Plants

USD U.S. Dollar

WDN Water Distribution Network

WP Wilting Point

Part I

Introduction

Chapter 1

Background

The theme of this dissertation is the optimal layout, design and dimensioning of water distribution networks (WDN) for irrigated agriculture: a way towards efficient investments in the irrigation sector. A WDN is an essential public infrastructure in the supply of water for irrigation or domestic uses. The construction and operation costs of such networks are very high and an optimal design of the WDN can produce enormous savings in construction and recurrent energy costs for the implementation agency. This is especially important in the context of developing countries. Many public WDN and irrigation systems in developing countries suffer from physical, managerial, and financial problems and are a serious drain on public funds. Many have broken down or operate at very unsatisfactory levels, or land just goes out of production (World Bank, 1994, 2002). These problems can have many origins, for example bad appraisal of the investment, bad dimensioning and oversizing of the WDN.

As Repetto (1986) pointed out, development aid and the financing policies behind many such investments have contributed immensely to this problem. Repetto's seminal work argued, at that time, that neither international donors, nor national irrigation agencies are accountable for the invested capital at risk. This has induced a very high demand for investments in new public WDN and irrigation projects. Public finance in many such countries was not enough to cover the operation and maintenance activities. Many of those irrigation schemes underperformed, corruption took over, and inequitable water allocation, dissatisfaction of farmers and low cost recovery were the consequences. This type of development aid and financing policy is prone to moral hazard and the demand for investment in oversized WDN and irrigation schemes. The decision-maker either does not rely on a cost-benefit analysis (CBA) at all, or when it does, the CBA becomes biased towards acceptance. Ex-post analysis of such investments can be performed only after a long time. Only then can initial assessments be finally compared with real numbers and experiences.

These problems are well known and not exclusive to the irrigation sector, but affect the whole lending policy of donors. In late 1992, the influential Wapenhans Report (an internal review of the performance of World Bank projects) delivered a devastating picture of the lending procedures of the Bank and of the failure of most of the projects initiated since the beginning of the 1980s. It was reported that 37.5 % of the Bank's projects completed in 1991 were failures, the most affected sectors being agriculture, and water supply and sanitation (World Bank, 1994). The Wapenhans Report was a milestone for rethinking the World Bank's lending policy on the agricultural and irrigation sector. More recent internal studies of the World Bank on the application of CBA for project appraisal confirm this picture. The Bank reports that: "...the percentage of projects with such an analysis dropped from 70 percent to 25 percent between the early 1970s and the early 2000s" (World Bank, 2010). The level of investments has dropped drastically ever since in the sectors of irrigation and drainage, with a dramatic drop in annual investment from $1.0 billion and $1.2 billion in the 1980s and early 1990s, to an historical low of $220 million in 2003 (World Bank, 2005). Given the fact that the world population is continuing to increase at a very high rate, it is obvious that the agricultural and irrigation sectors will need support to cope with the increased demand for food in the future. The Food and Agricultural Organization (FAO) estimates that, by 2030, food production needs to grow at a 1.4 % yearly rate to meet the demand and that half of this growth should come from irrigated agriculture (World Bank, 2005). The World Bank has realized the problem of the sharp decrease of investments in the agricultural sector in the last decades and the implications for food security. A re-engagement of the Bank in the agricultural sector was a natural reaction to these problems, and concrete strategies were designed in an effort to avoid past mistakes, aiming at a more efficient lending policy and investment success in the agricultural sector in general and irrigation in particular. These new strategies in the agricultural sector found expression in works such as, the Agriculture Investment Sourcebook (World Bank, 2004) and other publications guiding the World Bank's new corporate strategies (World Bank, 2005, 2010a).

Investments in WDN and irrigation will hopefully be made under much more comprehensive criteria and methods. This dissertation aims to contribute to this process.

Chapter 2

Problem domain and literature review

2.1 Introduction

This work deals with the investment and dimensioning analysis of branched Water Distribution Networks (WDN) for agriculture through the use of mathematical programming optimization methods. The optimization of WDN's (branched and looped networks) looks back to a body of knowledge from more than 40 years of research. The WDN design in its standard optimization form consists of dimensioning the pipeline diameters of the network by minimizing the investment costs while obeying the hydraulic constraints of the system. If the system is powered by a pump and the energy costs of pumping water should be included, the objective function is non-linear. Because commercial pipeline diameters should be used, the decision variables are discrete in nature. Furthermore, the hydraulic constraints of the problem are also non-linear. These facts make the dimensioning of WDN a very difficult combinatorial optimization problem of the Mixed Integer Non-linear (MINLP) type[1].

[1] This is also the case for the branched WDN considered in this work, where the irrigation schedule is an endogenous variable, i.e. flows cannot be determined *a priori*.

This chapter is structured in two sections. In Section 2.2 the problem domain is presented with a standard formulation. Section 2.3 introduces the current state of knowledge regarding the dimensioning of WDN. The review is intended to be representative rather than exhaustive. It will discuss relevant corner-stone methodological developments, setting them in an historical perspective whenever possible. A summary of research gaps for real-world dimensioning of irrigation WDN will close the chapter.

2.2 Problem domain

The main problem in dimensioning WDN for agriculture is the difficulty in linking economic reasoning towards justifiable investments with reliable engineering designs obeying the necessary hydraulic and irrigation management conditions. Most of the work done in dimensioning WDN is based solely on minimizing the costs of the network, i.e. the initial investments on pipelines and pumping system, and recurrent energy costs for non-gravity systems. An integrated design based on an in-depth economic-agricultural engineering analysis, considering the achievable benefits of such investments, is mostly not addressed. The balance between life-cycle benefits, life-cycle costs, initial investments and the implications this balance can have on the optimal design of a WDN are completely side-stepped. Avoiding this complex but necessary analysis can lead to oversized WDN, too high operation costs, low energy cost recovery, and needless to say, impossible recovery of the invested capital. A re-thinking towards simultaneous economic appraisal and dimensioning of WDN investments for agriculture is imperative.

Most of the difficulties in the development of more realistic and economically funded methodologies for full appraisal of such projects, rest on the mathematical difficulty of the problem. As mentioned in the introduction, we are concerned with a combinatorial optimization problem of the MINLP type.

A general formulation of the design problem with discrete diameters for a branched network is normally set-up as an minimization problem in the form:

Minimize: Total Cost = initial investment costs + operation costs (non-linear)

Subject to:

(a) The hydraulic law of mass conservation;

(b) The hydraulic law of energy conservation (non-linear);

(c) Minimum nodal head requirements.

The objective function can be represented in a simplified mathematical form by:

$$Min.\ total\ cost = \sum_{ij}\sum_{k} I_{kij} \cdot c_k \cdot L_{ij} + g\left(Q_{pu}, P_{pu}\right) + \left[\frac{\rho g \cdot Q_{pu} P_{pu}}{\eta}\right] \cdot Thrs \cdot p_e \qquad (2.1)$$

where (i, j) are the sections of the network and the three aggregated sums are: (1) the pipeline investment costs, where I_{kij} is a binary indicator variable equal to one if the pipeline of type k should enter the solution and zero otherwise ($\sum_k I_{kij} = 1\ \forall ij$); c_k represents the unit cost of pipeline type k and L_{ij} the length of the pipeline at section (i, j); (2) a function for the pumping system cost $g\left(Q_{pu}, P_{pu}\right)$, which is dependent on the pump's discharge Q_{pu} and head P_{pu}; (3) the energy costs of pumping water, i.e. a nonlinear relationship in the bilinear term discharge and pressure of the pump $\left(Q_{pu} \cdot P_{pu}\right)$. The parameter ρg is the specific weight of water, η the overall power efficiency, $Thrs$ is the total operation hours per year and p_e the energy cost.

The above objective function is minimized subject to the following constraints[2]:

The law of mass conservation, which demands that at each node the total inflow be equal to the total outflow:

$$S_{ij} - \sum_k Q_{ij} = 0; \ \forall ij \tag{2.2}$$

where S_{ij} is the total nodal supply or inflow and $\sum_k Q_{ij}$ the total node outflow in section (i, j).

The law of energy conservation, which states that the difference between the piezometric head at the entrance $(P_i + z_i)$ and end edges $(P_j + z_j)$ of section (i, j) is given by the head loss due to friction in the pipeline[3]:

$$P_i - P_j + z_i - z_j = \frac{10.68 \cdot L_{ij}}{C_{HW}^{1.852} \cdot I_{kij} \cdot D_{kij}^{4.87}} \cdot Q_{ij}^{1.852} \ \forall (i, j) \tag{2.3}$$

The right side of the above equation shows the Hazen-Williams[4] equation that empirically relates the pressure drop due to friction in the pipelines to flow (Q_{ij}), pipe type k diameter (D_{kij}), pipe length (L_{ij}) and an appropriate roughness coefficient (C_{hw}) dependent on the pipe's material.

The last constraint of this general formulation assures fixed minimum operating pressures at the withdrawal nodes:

$$P_i \geq P_i^{\min} \ \forall i$$

In the optimization model proposed in this dissertation for branched WDN, the irrigation schedule and number of irrigation hydrants operating simultaneously in each branch

[2] The hydraulic foundations and settings are discussed in Chapter 5.

[3] Minor losses due to pipe bends, valves etc., are not accounted here. A comprehensive discussion of the hydraulic characteristics of the network is given in Chapter 5

[4] The Hazen-Williams equation will be discussed in Chapter 5.

are considered endogenous. This implies that the pipeline discharges of the network cannot be fixed *a priori*. This fact, together with the discrete nature of the decision variables (diameters), the non-linear objective function and non-linear constraints, define an optimization problem of the Non-linear Mixed Integer Programming type (MINLP). These WDN problem types are very difficult to solve by classical optimization algorithms and are classified as being of *NP-hard* time complexity (Yates et al., 1984). The denomination *NP* refers to a classification to order the difficulty of solving the problem at hand (computation time). NP means solvable by a Non-deterministic *Turing machine* in Polynomial time[5]. Furthermore, if the problem belongs to the NP-hard type (an NP-completeness class), there is no known polynomial-time algorithm to solve these problems (Nemhauser and Wolsey, 1999). Nevertheless, as Wolsey (1998, p. 88) states:

> "...in spite of the *NP*-completeness theory, using an appropriate combination of theory, algorithms, experience, and intensive calculation, verifiable good solutions for large instances can and must be found".

Given the MINLP nature of the WDN dimensioning problem, researchers have tried to find simplifications or mathematical innovations to bypass difficulties caused by the discrete, nonlinear nature of the problem. In the process, many important real-world problem components are disregarded or side-stepped. A good model though, should be suitable for real-world applications and additional necessary complexities should be included.

The difficulty of this applied problem has motivated a broad spectrum of different approaches in the literature, which will be discussed hereafter.

[5] Schrijver (1986, p. 17) explains it as:
"...for deriving the polynomiality of an algorithm which performs a sequence of elementary arithmetic operations, it suffices to show that the total number of these operations is polynomially bounded by the size of the input, and the sizes of the intermediate numbers to which these elementary arithmetic operations are applied are polynomially bounded by the size of the input of the algorithm."

2.3 Literature review

When reviewing the literature in dimensioning WDN, it is useful to distinguish between looped and branched WDN. The looped water distribution networks are far more commonly used for domestic and industrial purposes, while the branched WDN are more often used in the rural context, especially for irrigation (Bhave, 2003). The problems show different mathematical complexities. If it is possible to establish fixed demand patterns, the flow directions in the branched systems are uniquely defined. In the looped systems it is not possible to define concrete flow directions *a priori* and the problem turns out to be more complex. The redundant looping paths give the looped system a higher level of operational reliability but come at a higher initial investment cost. This is the main reason why these systems are not frequently used in irrigation systems (Bhave, 2003, 2006). The body of research is larger for looped than for branched WDN; because the looped WDN methods are also suitable for branched systems, it was decided to review the literature for both cases.

Branched WDN:

Karmeli et al. (1968) is the most common reference for the initial work on optimizing branched WDN. The authors use Linear Programming (LP) for discrete-pipeline diameter and pump head dimensioning. In these early approaches the discharges $q(n)$ at each delivery node $n \in \{1, ..., N\}$ were assumed known and the discharges across the pipeline sections attached to each node $Q(n)$ could be readily calculated by the law of mass conservation. Because the discharges were considered fixed, the head losses in the pipelines could be calculated *a priori* for each candidate pipe and respective diameter through a modified Hazen-Williams equation expressing head losses J in $\mathrm{m} \cdot \mathrm{kmm.}^{-1}$

$$J = 1.13 \cdot 10^{12} \left(\frac{Q}{C_{hw}} \right)^{1.852} \cdot D^{-4.87}$$

In the above equation the discharge Q is given in cubic meters per hour, D is the pipe diameter in millimeters, and the C_{hw} friction coefficient dependent on the pipe's material. For each section and demand node n there are $G(n)$ different pipes to be considered. A tuple (n, j), refers to j^{th} pipe diameter in the n^{th} section of the network. The head loss $h(n)$ for each candidate diameter is calculated as the product of the unit head loss parameter $J(n, j)$ multiplied by the variable length $X(n, j)$ for each alternative pipe j. The total head of the system is given by $E(0)$ and the cost assumed to be $k(0)$ per unit. The authors refer to $k(0)$ as a cost factor per head unit of the pump that includes the annual operating cost and capital costs of the pump expressed as an annual amount by a suitable capital recovery factor.

The model was set-up as to minimize the total costs K, where $k(n, j)$ is the pipe unit cost and $X(n, j)$ the length of the pipe j in section n:

$$K = k(0) \cdot E(0) + \sum_{n=1}^{N} \sum_{j=1}^{G(n)} k(n, j) \cdot X(n, j),$$

subject to the condition that the sum of chosen pipe lengths for each section must equal the total length of that section, subject to the minimum operation pressures at each demand node and also subject to the usual non-negativity constraints.

The decision variables of the model here are the pipe lengths only. The assumption of constant flows in the pipelines side-steps the problem of nonlinear constraints for accounting head losses in each pipe. The problem can thereafter be solved by Linear Programming (LP). For each section of the network, alternative pipes with different given diameters, friction loss factors and price per length unit are given as alternatives to choose from. These are chosen by minimizing the total cost of the network. Karmeli et al. (1968) state that the optimal solution will show at most two different pipe diameters for each section.

The first formulations for designing branched WDN with LP (for more than 40 years) had, of course, strong assumptions and limitations. In particular, the strength of today's computers was not available at the time, which hampered the authors from considering more realistic features. Nevertheless, the work of Karmeli et al. (1968) is probably the most adequate reference for exposing the evolution in the design of WDN. Many real-world features could not be included in the design at that time, and the model had several shortcomings:

1. All nodes need to be irrigated at the same time, i.e. the potentials for optimization are not exhausted. Considering different irrigation shifts would reduce the design discharge of the pump and lower the pump and operation costs (energy) of the investment.

2. The pump head is calculated for only one discharge, no variable loadings allowed, for example, there is no consideration of irrigation shifts, this means that division of the whole irrigation area into irrigation sub-units is not possible. Furthermore, the pump head is calculated for the most unfavorable node, i.e. nodes near the pump source will have too high a pressure.

3. The model can only be used for flat terrains. Considering topographical elevations demands an energy balance for each pipeline and is not possible with the taken formulation of the unit loss gradient in $J(n, j)$, which actually allows having the pipe lengths as decision variables and not the diameters.

4. The pump cost $k(0)$ was assumed to be a linear function of $E(0)$, calculated only for one load of the system.

5. There is no analysis of the trade-offs between capital and energy costs.

6. The layout and extension of the network is fixed, no cost-benefit analysis of the different irrigation hydrants.

Calhoun (1971), and Robinson and Austin (1976), developed the basic model further by including pipe pressure specifications, i.e. the inclusion of maximum head constraints for pipes in the model. The authors also improved the linear formulation of Karmeli et al. (1968) for the pump linear cost formulation $k(0)$, developing an iteration procedure solving a nonlinear relationship of the pump costs with the head at the source calculated iteratively. The authors also introduced pressure reducing valves limiting the problem of too high a pressures in the nodes near the source.

Liang (1971) implements a Dynamic Programming (DP) approach for the dimensioning branched WDN, not only minimizing the costs of pipes, but also the energy costs. In the DP formulation each pipeline section is considered a stage, and state variables were defined as the pressures at the pipe inlet and outlets. The diameters of pipes were taken as discrete decision variables. The relationship between the input and output state variables (pressures) is established by the relationship between the inlet and outlet pressures of each pipe. The state (head) changes were based on the Darcy-Weisbach friction loss equation.

Swamee et al. (1973) present a nonlinear approach for continuous diameters, that transforms the original formulation to an unconstrained problem combining the objective function and constraints in a *merit function* using Lagrangian multipliers. The solution provides optimal pipe diameters, pumping head, hydraulic gradient line, and the minimal cost of the network.

The problem with nonlinear approaches is that they cannot guarantee to find the global optimum of the problem. Furthermore, the author considers continuous diameters. These solutions have to be rounded for real-world applications (commercial diameters of pipelines); therefore the optimality is lost.

Bhave (1978) presents a continuous-diameter non-computer approach for optimizing the diameters of the pipelines including the elevation heads (topography) of the different nodes in the calculation. The procedure initially selects the piezometric heads (node elevations plus demanded head) through the critical path method (Bhave 1978, 1979). These heads are iteratively corrected through a cost-head loss-ratio criterion until the optimal global solution is obtained. The procedure solves the problem for continuous diameters, when minimizing the total costs. The global optimum is again lost when converting the diameters to discrete commercial sizes. The non-computer approach is not suitable for large networks.

Young (1994) claimed that previous studies had not taken account of minimum operating heads at interior nodes of the branched networks, that dimensioning on uneven terrains was not possible with former methods. The author used Non-Linear Programming (NLP) and Lagrange Multiplier methods applying the Newton-Raphson method for solving the nonlinear equations. The major drawback of the NLP approaches is that calculations which are made for the continuous space and pipeline diameters need to be adjusted for commercial pipeline sizes. The procedure can render solutions unfeasible when using round off diameters. Furthermore, Johnson et al. (1996) severely criticize the approach of Young (1994) contradicting not only the results but most of all the statement that former methods could not be applied on uneven terrains. Johnson et al. (1996) show that such an NLP procedure is not necessary for solving the branched problem under given discharges and nothing significant has been added to the effectiveness of formulations by former research.

Labye (1981) and Labye et al. (1988) addressed the one loading problem by developing an iterative procedure that allows the consideration of several regime flows when dimensioning the pipelines. The procedure starts by attributing to each section of the network the minimum commercial diameter possible, subject to the maximum allowed flow velocity in the pipe for a calculated discharge. For this initial set of diameters the piezometric head at the upstream is calculated that satisfies the minimum allowable head given the most unfavorable outlet point. The final solution is obtained by iteratively reducing the above calculated piezometric head until the available head at upstream. This is done through selection in each iteration of the section for which an increase in pipeline diameter gives the lowest increase in the network cost.

Bhave and Lam (1983) criticized the common procedure of optimizing the dimensioning of a WDN based on given layouts, arguing that the layout is in itself an optimization problem. They considered for the first time the layout problem together with the problem of dimensioning the pipelines. The authors developed a modified *Steiner Tree* approach that also takes in consideration the nodes discharges and available Hydraulic Gradient Levels (HGL) at the source nodes and at the demand nodes (minimum demanded). The problem is solved by an unconstrained iterative nonlinear model. The authors show that additional junction points in the network are necessary for designing the minimum cost layout. These junction points normally rest within the triangle formed by three nodes and can eventually coincide with one of the triangle nodes. Nevertheless, the approach makes strong assumptions. It works with known fixed demand patterns at the nodes and also fixed HGL. The strongest assumptions are the continuous diameters. There are also no topographical considerations regarding the previously calculated HGL.

Goncalves and Vaz Pato (2000) also follow the approach of integrating a solution for the optimal layout and hydraulic dimensioning of a branched WDN. Their work is based on a three module procedure. In the first module the Steiner tree problem is solved for the shortest path applying a heuristic procedure developed by Takahashi and Matsuyama (1980). In the second module the pipeline discharges are calculated based on probabilistic on-demand calculations based on Clement and Galand (1979). After the discharges for each pipeline component or network are calculated (fixed), Mixed Integer Linear Programming (MILP) model solves the problem of determining pipelines diameters, pump locations and heights (booster pumps are also considered). The objective function includes the initial investment costs in the pipelines and pumps and also energy costs. The procedure can be applied to branch networks where on-demand dimensioning can be justified. By fixing the water demands with the help of probabilistic methods the authors side-step the problem of varying loadings in the network. The application of the MILP model is thereafter straightforward because the non-linearity in the constraints and objective function are no longer effective (fixed discharges) and can be made linear. The procedure considers different pipe pressure classes accomodating the maximal pipe pressures allowed. The model would not be adequate for a rotational water distribution, where the optimal schedule of operating opening valves (and with it discharges in each irrigation shift) should be considered endogenous.

Lejano (2006) also addresses the layout problem but considers two new aspects: (1) the layout and discrete-diameter dimensioning are optimized simultaneously, and not sequentially like the procedures before (he departs, nevertheless, from a given layout, where links are dropped or kept with the help of binary variables); (2) the benefits generated by water in each irrigation demand node are included in the objective function. The demand nodes are only included in the layout if their marginal contribution to the net-benefits in the objective function are at least as large as their marginal costs. In this way Lejano responds to the quest raised by Walski, (2001) on the disadvantages of concentrating research only on minimizing the networks' pipeline costs. However, the work does not consider any water distribution procedure, either on-demand or rotational. The water demands are given and occur simultaneously at all nodes. The pipeline discharges serving the nodes are calculated through conservation of mass. Water demands need to be served at the same time in the model, i.e. no scheduling. This fact is extremely disadvantageous for real irrigation applications.

The following authors modeled irrigation systems at the farm-level. Although here we are discussing different scales, the ideas are seminal for the purpose of the present work and the optimization procedures are similar to the ones needed when designing higher scale branched WDN for irrigation purposes.

Holzapfel et al. (1990) proposed a nonlinear programming model (NLP) for dimensioning a drip irrigation system at the farm level (a real-world application). The model's objective function balances the benefits of irrigation through an empirical crop production function dependent on water and the costs of implementation and operation of the system (energy costs). The integration of water production functions in the objective of the model impacts on the design. Water will be delivered to the plants according to the balance between yield profits and systems costs. This means the dimensioning depends very much on the current cropping pattern, and further, is not made for the peak water requirements of the plants (this could eventually be problematic if cropping patterns are to be changed).

Another interesting feature of the model is the determination of the total number of sub-units in the system and the number of units to be operated simultaneously. The model results show that when maximizing profit (instead of minimizing the costs of the pipelines and operation, as is usual) the resulting pipe network does not correspond to the minimum cost network. The resulting diameter of the lateral (which represents the largest amount of pipe in the network) is larger than the minimum commercial available diameter. The main conclusion is that operation and dimensioning irrigation systems should be designed on a net-benefit maximization basis and not only by minimizing costs. Although this model is applied at the farm-level the lessons can be used for dimensioning WDN at other scales. The size of the network is not addressed in this work; it would be useful to see the results of balancing benefits and costs on the size, dimensioning, and operation management.

Dandy and Hassanli (1996) also apply a nonlinear optimization model for determining the optimal partitioning of a drip irrigation system in sub-units. The model departs from an assumed fixed layout of the pipeline system. Their model only considers minimization of pipeline costs and discounted energy costs. The model is applied for a flat terrain and based on a complete enumeration for the drip and multiple sub-unit system. The authors conclude that the optimum size of the sub-units depends strongly on the trade-offs between the pipe costs (diameters), the corresponding cost of energy, and the number of irrigation shifts. The model identified the minimum dimensions of the irrigation sub-units for a different number of irrigation shifts. The optimal extension of the system is not addressed.

Lamaddalena (1997), Lamaddalena and Sagardoy (2000), and Pereira et al. (2003) developed methodologies for dimensioning pipelines in irrigation networks responding to the quest of including multiple flow regimes. The procedures also include a series of performance indicators for evaluating the design. These approaches differentiate from former research on dimensioning irrigation networks based on simple probabilistic calculations of expected water demand, which resulted in using only one flow regime for dimensioning. Former dimensioning methods did not consider simulations of the effects of possible configurations of nodes operating simultaneously, or changing cropping patterns and managements, i.e. different flow regimes, demand hygrographs. The consequence can be harmful hydrant/node pressure drops in the times of peak demands reducing performance of irrigation. The proposed methodology considers the simulation of different flow regimes based on configurations of outlets/nodes operating simultaneously in the system.

Daccache et al. (2010) further worked on this field tackling the problems of dimensioning on-demand irrigation networks, most specifically the high variation that the upstream head of the network can be subjected to, when the number of nodes operating simultaneously shows strong variations. The work considered the design optimization of pressurized systems (sprinkler systems) by using comparable methodologies to Lamaddalena (1997) and Pereira et al. (2003), i.e. (1) analysis of the on-demand water distribution system; (2) pipe sizing of the on-farm network by applying the Iterative Discontinuous Method (Labye, 1981); (3) network solution through hydrant characteristic curves; (4) performance analysis of the sprinkler network.

Looped WDN: Introducing the large body of research literature on dimensioning looped WDN is no easy task. Nevertheless, it seems appropriate to start with the work of Alperovits and Shamir (1977) given the benchmark character this work has had for the development of modern methods of dimensioning looped WDN.

Alperovits and Shamir (1977) introduced the Linear Programming Gradient approach (LPG) method applied to a looped network. In the method each link can be composed of different pipes with different diameters. The iteration starts by considering an assumed flow distribution. Given this distribution, an LP formulation of the problem is used for determining the optimal diameters considering constraints on minimum heads at the demand nodes. In the next step a gradient method improves the cost of the network by changing the flow distribution. The direction of change is guided by the dual formulation of the hydraulic constraints, where flow changes are made that induce a negative gradient of costs. The method does not allow the introduction of nonlinear cost components.

Fujiwara and Khang (1990) address this knowledge gap and introduce the Non-linear Programming Gradient (NLPG). A nonlinear objective function is established that includes the formulations for energy costs previously missing. Altogether the objective formulation includes capital costs and operating costs, and is expressed as a function of pump heads, flows, and head losses in the arcs of the network.

Eiger et al. (1994) show that the former researchers did not properly consider the non-convexity of the minimization function, forgoing the search for the global optimum and taking local optima as solutions. They introduce a method that finds tight lower bounds to evaluate the quality of the solutions found (local optima) and introduce an algorithm by Schramm (1989) that can handle the non-convexity of the objective function (the bundle-trust algorithm). The bounds on the objective function are determined using dual theory.

Sherali et al. (1998) and Costa et al. (2001) further improved the work of Eiger et al. (1994) by developing, respectively, a better lower bounding scheme and better convergence sequences between upper and lower bounds.

Bragalli et al. (2012) use, most probably for the first time, a 'direct' MINLP approach for dimensioning a WDN. Their method is based on a non-convex continuous NLP relaxation and an MINLP search. The major contribution of this work to mathematical programming for WDN is the development of special techniques for dealing with nonlinear and discrete parts of the problem, allowing the use and further development of MINLP available solvers. In fact, the authors develop techniques for dealing with the non-convexity in the constraints of the model, which were later incorporated in the open-source MINLP software Bonmin (Bonmin, 2006). Innovative was also the derivation of a continuous objective function by fitting a polynom to the discrete input cost data, and the development of a smooth (approximate) relaxation of the head loss in the pipelines based on the Hazen-Williams equation.

Given the inherent difficulties of using closed mathematical programming methods, researchers started to interface the optimization algorithms with hydraulic simulation models for verifying the hydraulic constraints.

Hossein and Mottaghi (2006) apply a MILP model to the dimensioning of looped WDN, where the mathematical programming formulation interacts with a hydraulic simulation model. In their model the commercial pipeline diameters and pumps, as well as their respective costs, are chosen through binary variables. The MILP model only includes pressures and velocity constraints in the pipelines. The mass and energy conservation constraints are dealt with externally by the hydraulic simulator. The optimization model considers only single loading conditions interfacing with the hydraulic simulation model through an iterative process. At first the pipeline diameters and demanded power are assumed based on commercial availability. The hydraulic model takes on the starting values and simulates the nodal pressures, pipeline discharges and the head losses in the system. The calculated flow discharges are set in the optimization model and new diameters are calculated. The diameters are thereafter compared with the original assumption; if they are the same after several iterations the algorithm stops, if not, the optimized diameters are introduced in the hydraulic simulation model and the new discharges calculated until the procedure converges.

From this line of thought (interfacing optimization algorithms with hydraulic simulation) a new body of meta-heuristic optimization methods for WDN has emerged.

The **Meta-heuristic Optimization Methods** try to circumvent the difficulties of the classical mathematical programming models when dealing with complex MINLP and other hard mathematical formulations. In the case of optimizing the design of WDN, the solution search algorithms are always interfaced with hydraulic simulators e.g. EPANET (Rossman, 2000) to validate the hydraulic constraints of the problem. In this way the difficulties due to non-linearity's in the closed mathematical programming methods are avoided. The methods are based on heuristic iterative search algorithms, which are likely to find a very good solution for the problem. There is, nevertheless, no guarantee that the solution found by these methods represents the global optimum (Hillier and Lieberman, 2005). Another disadvantage is the complex parameter settings for each new application. As Hillier and Lieberman (2005, p. 617) state: "...heuristic methods tend to be ad hoc in nature. That is, each method usually is designed to fit a specific problem type rather than a variety of applications".

In recent years meta-heuristic methods have been developed strongly, trying to provide a more general structure to heuristics and developing search strategy guidelines for developing heuristic search methods. The meta-heuristic optimization methods can be differentiated according to the search strategy as follows.

(1) **Local Search** heuristics, where search departs from a single initial trial solution iterating in its neighborhood, unfolding a trajectory through the search space until a certain stop search criteria is achieved. The heuristic very much depends on the choice of the starting point for performing the search. In the presence of multiple local optima the procedure can be caught in local optima away from the global optimum.

(2) **Population-based meta-heuristics,** which are procedures that randomly generate a set of initial solutions (initial population) as opposed to departing from a single solution. The population of individuals (solutions) is "genetically" modified, through specific operators, e.g. recombination and mutation operators. The algorithm iterates, creates new generations, by further applying operators to the group of solutions until a stop criteria is achieved e.g. certain number of generation, etc.

(3) **Constructive meta-heuristics**, which start with an unfinished solution and sequentially add solution components e.g. different pipes, building a complete solution to the problem. Several solutions are built in this way, forming a tree of constructive decisions.

For a more comprehensive description of the many different types of Meta-heuristics see Boussaid (2013) or Soerensen (2013b).

The main characteristic of these meta-heuristic optimization methods are the analogies to some of Nature's specific processes, which are borrowed for the meta-heuristic optimum search strategies.

The **Tabu Search** meta-heuristic is a local improvement procedure. Nature's analogy of the method is based on the human memory process (Glover and Laguna, 1997). The strategy of the Tabu Search method for escaping local optima is to allow for non-improving solutions in moving forward in the search. When departing from the last local optima only the mildest non-improving steps are taken. If, in the course of further iterations, there are ascent steps the steepest is taken. The main characteristic of the Tabu Search is the danger of returning, or cycling back, to the same local optima. To avoid this problem the method incorporates a kind of "memory" called Tabu List that rules out the paths towards the former local optima. The length of the Tabu List is ruled by the Tabu Tenure parameter (Glover and Laguna, 1997; Hillier and Lieberman, 2005; Soerensen, 2013b).

The **Simulated Annealing** meta-heuristic is like the Tabu Search, a local solution improvement procedure. The natural process analogy is with the physical annealing of crystals to low energy states (Aarts et al., 2005). In this process temperature plays the major role. Temperature is increased to increase the flow of molecules. After the temperature is slowly decreased the molecules rearrange themselves randomly arriving at the lowest energy state, corresponding to the flawless crystalline structure (Cunha and Sousa, 2001; Soerensen, 2013b). The novelty of simulated annealing lies in the strategy to "escape" from current local optima to an immediate neighborhood, the ability to accept worsening the solution and move with the search finding other local optima. The algorithm is based on a probabilistic strategy to move forward in the iterations, escape local optima. Each candidate neighborhood solution is equated with an acceptance probability according to $Prob\{acceptance\} = e^x$, where $x = \dfrac{(Z_n - Z_c)}{T}$, and Z_n is the current candidate and Z_c is the current trial solution. The parameter T is called the "temperature" in analogy to the physical annealing process and controls the probability of acceptance of worse solutions.

The “temperature” is time dependent and set at a high value at the beginning of the search process. This allows the search to extend to broader areas of the solution region. Lowering the “temperature” will reduce the probability of acceptance of worse solutions. The search process gradually increases the concentration on better solutions, by diminishing the “temperature” and rejecting more and more proportions of moving steps that produce worse solutions (Cunha and Sousa, 2001; Hillier and Lieberman, 2005; Soerensen, 2013b).

Cunha and Sousa (1999) apply a simulated annealing heuristic for a least-cost design of a looped WDN, using the Hazen-Williams equation for accounting head losses. Unknowns were the flow rates, their directions and pipeline discharges. The hydraulic equilibrium states are simulated externally by the Newton-Raphson method. If the hydraulic constraints are satisfied the candidate solutions are tested with the above discussed stochastic criteria and the process iterates further. This means, the algorithm stochastically chooses new configurations of the network in the neighborhood of the current solution, which differ from the current solution by the diameter of only one of the network pipes. Cunha and Sousa compare their results to the WDN benchmarks in Alperovits and Shamir (1977) and to the Hanoi Network by Fujiwara and Khang (1990). The work is one of the pioneering applications of simulated annealing to the WDN dimensioning problem. However, an aspect missing in the analysis is the consideration of variable water demand patterns. Furthermore, these works are applied only to gravity networks; the problems inherent to nonlinear energy costs in the objective function are, of course, not addressed. The influence these factors can have on the energy consumption (the highest life-cycle cost of the WDN) and on the optimal design of the WDN are undeniable and need to be included in further SA and other meta-heuristic research. Another issue related to this line of research, is the typical insecurity of heuristic methods regarding the global optimum. In this type of algorithm there is no guarantee that the solution found is really the global maxima of the problem (Hillier and Lieberman, 2005).

Cunha and Ribeiro (2004) address this problem and compare the results of the annealing approach in Cunha and Sousa (1999) to the Tabu Search (TS) procedure using the same benchmarks. The results of the Tabu Search Method show improvements for some of the networks in Cunha and Sousa (1999). The main message of this work though is that no real conclusion can be drawn regarding the comparison between SA and TS, or even with other meta-heuristics given the small number of case studies available. Although results are motivating doubts about the global optima still persist.

Genetic Algorithms (GA) belong to the population-based meta-heuristics**.** The Nature metaphor is derived from the biological theory of evolution developed by Charles Darwin. Concepts like variation, natural selection, mutation, or the survival of the fittest are borrowed from the evolution theory by the GA optimization approaches. For casting the optimization problem, the different solutions are transformed to binary code strings (an analogy to genetic coding). The GA draws on evolutionary principles for efficiently searching the solution space. New trial solutions ("children") are found by combining "parent" solutions through binary code cross-over, and mutations processes guided by probability decision rules.

In the principle of survival of the fittest, a child that inherits better chromosomes and genes from their parents is more likely to survive and pass these features to the next generation. The genes of the chromosomes can be understood as the 0/1 digits in each binary code. Departing from a predefined population of feasible solutions or parents (the hydraulic equilibrium states are again simulated externally and feasibility for each candidate solution controlled), the fittest members are chosen. The fitness of a solution refers to its value in the objective function. These parents are paired randomly and each gives birth to two children (new feasible trial solutions). The features of the children's chromosomes are given by a random mixture of parent's genetic information, e.g. a cross-over matching of the binary codes of the two parent solutions. Children can also show mutations in any gene (the 1/0 "genes" in the binary code of each solution). The mutation's occurrence is ruled according to a predefined probability. If a random mixture of parent's features and mutations produce an unfeasible solution this is named a miscarriage and discarded, the process goes on until a feasible solution is found. When casting a GA to dimensioning a WDN, the different pipe diameters are attributed binary codes. If the network has 10 pipeline sections, and there are 16 different diameters which can possibly be attributed to each section, the binary string of a candidate solution will consist of 10 sets of 16 different 4 bit strings ($2^4 = 16$), each 4 bit string representing a diameter.

Kadu et al. (2008) present a GA for WDN dimensioning that proposes some modifications to the standard GA, reducing the search space. Kadu et al. (2008) introduce improvement features for the typical GA performance parameters: search space (population size), coding scheme, penalty method, fitness function, selection and cross-over parameters and probability of cross-over. The authors' most determinant contribution to GA for dimensioning looped WDN was surely the introduction of the critical path method (Bhave, 1978) for reducing the solution space. For one of the benchmark problems at hand Kadu et al. report that using the proposed improved GA achieved a dimension reduction to 0.074% of the entire search space. Also worthy of mention is the adoption of real coded strings after Vairavammoorthy and Ali (2000) instead of binary strings for reducing computational time. The authors introduce a penalty function for discouraging minimum pressure violations into the objective fitness function.

Kang and Lansey (2012) apply a GA to a real-world WDN dimensioning problem. The authors consider not only the main trunk-lines of the principal water distribution system; they also integrated second-level planning scales, i.e. the transmission and distribution system to management areas. The authors criticize the current state-of-the-art in meta-heuristic methods for dimensioning WDN, arguing that the great majority of this literature has concentrated on new techniques, algorithms, and on the application of these stochastic search methods to simple benchmark WDN. These benchmarks are characterized by small sizes, i.e. using only a small number of pipelines (transmission or trunk-lines only) and not allowing a real-world suitability proof of the meta-heuristics.

The main goal of Kang and Lansey's (2012) paper was to move away from this benchmark study type, by integrating transmission-distribution system scales into the analysis. For this purpose the authors considered a network of 935 nodes, one source and one pumping station. Another innovation of this study was consideration of a pump system, i.e. the consideration of nonlinear energy costs in the objective function (fitness function). Furthermore, different demand loading conditions were also considered, avoiding the typical side-stepping of nearly all other studies applying stochastic search methods to the dimensioning of WDN. The minimum pressure constraints are assured by integrating a penalty in the fitness function.

These authors are most probably the first to apply a stochastic search algorithm (in the case of the GA) to a real world, complex and large WDN. Because of this, the authors were very concerned with the known shortcomings of meta-heuristic optimization, when it comes to prove the global optimality. Meta-heuristic algorithms cannot prove that a global optimum has been reached, these methods can easily be "caught" in local optima if the meta-heuristic is not trimmed enough for the complexity of the problem at hand. Because the huge majority of publications in meta-heuristics are only concerned with small benchmark models, this problem is never comprehensively addressed. For increasing the performance of their GA, these authors developed a heuristic for improving the initial population of solutions. They report improvements in avoiding typical meta-heuristic algorithm problems in the branched network parts of the system. In branched systems meta-heuristic models often produce inconsistent layouts, i.e. larger pipes downstream (Farmani et al., 2007). The EPANET toolkit (Rossman, 2000) is used here used to verify the hydraulic constraints, like in many other meta-heuristic algorithms.

The **Memetic Algorithm (MA)** also belongs to the population-based meta-heuristic approaches, drawing on GA principles, i.e. based on evolution. Like the GA algorithms they combine evolution procedures and local search techniques. The metaphor in this case is not biological but of a cultural evolutionary nature, the evolution of ideas, where "memes" are considered the unit of cultural evolution (Eusuff et al., 2006). Although MA algorithms are based on the same principles as GA, e.g. solution space, fitness measures for selection, stochastic combination of parents and mutations, they are more flexible than the "genes" in GA. As the opposite of genes, memes can be transmitted, e.g. through cross-over techniques like in GA, between any two individuals of the population, creating new candidate solutions. Moreover, new ideas (new candidate solutions) derived in the process of evolution can be immediately incorporated in other memes without having to wait for a next generation (Eusuff and Lansey, 2003; Eusuff et al., 2006).

Eusuff and Lansey (2003) apply an MA called Shuffled Frog Leaping Algorithm (SFLA), an MA metaphor based on "frogs" seen as hosts for memes containing memotypes or ideas (solutions to the optimization problem). The difference to GA is that ideas (solutions) can be exchanged between any individuals in the population and not only between parents and children forming new candidate solutions. The stochastic search algorithm is wider than the GA. Eusuff and Lansey (2003) also connect their algorithm to the EPANET toolkit (Rossman, 2000) for the hydraulic simulation. The fitness function is similar to Kadu et al. (2008), by using a penalty function for violations of the required minimum pressures. The benchmark WDN's are based on a gravity scheme, and again, the energy considerations and trade-offs between initial investment costs and energy costs are side-stepped.

Banos et al. (2007, 2010) follow a more pragmatic, less metaphoric, explanation of their MA algorithm. In this work the WDN layout and water demands are also considered as given. The MA algorithm initiates the search by randomly generating what the authors call agents i.e. network configurations, a reproduction procedure is applied, i.e. the starting network configurations are modified by applying variation operators for generating the "children" or new configurations. A local search optimizer is subsquently used to improve the original configurations, choosing from the different generated pipe diameters. One novelty in their MA formulation is the introduction of an entropy rate for determining the degree of similarity required for the combination of new "ideas" or trial solutions. They also test their model against the well-known Alperovits and Shamir (1977) and the Hanoi Water Distribution Network benchmarks. The authors compare their work not only with other meta-heuristic procedures (Simulated Annealing, Simulated Annealing combined with Tabu Search, Scatter Search and Genetic Algorithms) but also (and maybe the first real attempt) with a Binary Integer Problem (BIP) approach. The authors advocate the superiority of the developed MA comparing the results with these other approaches.

Although the MA approach developed in the Banos et al. (2007) work seems to provide very good results, the way the author uses the BIP approach for comparisons demands a better analysis and criticism. In their work the authors report that the BIP model delivered slightly worse results than the meta-heuristic approaches. This is surely true for this work, where the BIP has been implemented under quite disadvantageous conditions. The BIP is programmed in EXCEL using a built-in solver with quite strong limitations regarding the number of binary variables and constraints. Furthermore, and most importantly, the BIP model is run under the very limited number of iterations allowed by the built-in Excel-Solver (as mentioned by the authors). In the results the authors also report that the BIP shows the fastest solution times of all methods. This is not unexpected given the much reduced number of iterations allowed by the Excel solver. Given this, it is not surprising that the BIP results are worse, although only slightly worse, than the meta-heuristic methods. Having programmed the BIP in a modern environment like GAMS (Brooke et al., 2005) using a powerful solver like CPLEX (ILOG, Inc., 2013) would have been surely quite different. Another problem pointed out by Banos et al. (2007) regarding the BIP formulation is the intensive programming work. Using the above mentioned efficient modeling platforms, instead of Excel and Visual Basic, would have showed that the programming of BIP in such environments is by far less time intensive than programming meta-heuristic algorithms.

The harmony search algorithm (HS) belongs to the constructive meta-heuristic field. The heuristic is based on a stochastic search procedure that draws on music improvisation by musicians. It was proposed by Geem et al. (2001). In the HS algorithm each musician is taken as a decision variable; the musician generates or "plays" a note (value) towards the best harmony (global optimum). In reality a set of random solutions is generated (harmony memory), the process now iterates by randomly combining new solutions from the harmony set and from a random distribution. New solutions are recorded in the harmony set replacing the worst solution recorded at that time.

Geem et al. (2011) present an HS algorithm for dimensioning a WDN proposing an improvement for algorithm parameter setting: the Parameter-free Harmony Search (PSF HS) algorithm. The authors claim this reduces the known shortcomings of stochastic search algorithms, e.g. the necessity of tedious parameter setting. The authors further assess the general performance of standard GA algorithms, reporting that these heavily depend on the initial parameterization of population sizes, crossover and mutation operators (see also other authors on this matter, such as Gibbs et al., 2010). The improved HS is once again only applied to simple benchmark networks.

The development of meta-heuristic optimization models for the dimensioning of **branched irrigation WDN** has not experienced as much attention as the dimensioning of **looped WDN**. Nevertheless, some research work on these methodologies can be found for irrigation.

Farmani et al. (2007) apply a GA to the dimensioning of a branched irrigation network, where the objective function was set to minimize the investment costs through optimal dimensioning of pipelines and an optimal irrigation schedule (hydrant opening). The interesting feature of this work is the reference to the difficulties in applying meta-heuristics to practical irrigation problems. The problems reported with meta-heuristics for branched irrigation systems also refer to exhaustive parameter setting, i.e. the application of fixed and problem-independent operators typical in these methods, which will not result in much improvement compared with traditional closed mathematical-programming algorithms (Walters and Lohbeck, 1983; Farmani et al., 2007). The authors introduce modifications to typical GA operators addressing the combinatorial nature of the branched system. The authors report inconsistency of heuristic solutions selecting larger diameter pipelines downstream of each branch. In order to tackle these problems the authors changed the representation of solutions for two consecutive pipes, restraining the downstream pipe to have a lower diameter than the upstream one. Other modifications are introduced for the mutation operator, introducing perturbations for helping search for escaping local optima.

The authors compare two different water allocation strategies, rotation and on-demand water delivery. For on-demand water distribution the demand nodes are supplied in the same period with constant discharges, the decision variables are only the pipe diameters. In the rotation delivery case the decision variables are the pipe diameters and also the scheduling of the hydrant opening. The authors compare the results of the GA to an LP application. For the LP application to be possible in the rotational case, the authors introduce a quite daring trial-and-error technique for fixing *a priori* the opening schedule of the hydrants in each irrigation shift and only after optimizing the diameter of the pipes through LP. The MILP model of this dissertation calculates the opening schedule of hydrants endogenously using binary variables (a true optimization).

The systems addressed by the authors in their applications are served by gravity, i.e. the power costs of typical powered systems and their influence on the dimensioning are side-stepped. The introduction of power systems would surely demand for even more complex meta-heuristics, i.e. parameter and trial solutions tailoring.

This work has exposed some important problems that heuristic methods face when dealing with real-world applications and not relatively simple study benchmarks. The heuristics have to be adapted to every new problem at hand this gives each application a rather ad-hoc character regarding the complexity of algorithms developed by each single author. These algorithms change considerably according to the authors' metaphorical visions, their ways of trimming parameters, and also according to each different problem's complexity.

The strong development of heuristic or meta-heuristic optimization methods has produced an immense number of different algorithms, from Genetic Algorithms (GA), Simulated Annealing (SA), Tabu Search (TS), Shuffled Frog-leaping (SFL), Colony Optimization (CO) (Maier et al., 2003), Artificial Neural Networks (ANN) (Srinivasa and Brion, 2005), Harmony Search (HS), Cross Entropy (CE) (Perelman and Obstfeld, 2007), Scatter Search (SS) (Lin et al., 2007), Hybrid Algorithm (HA) (Geem, 2009), or Honeybee Mating Optimization (Mohan and Babu, 2010), just to mention a few.

More recently critical discussions have started doubting the usefulness of these never ending "creative" metaphors.

Weyland (2010) was perhaps the first critical voice regarding these *en vogue* metaphoric approaches, with a sharp scientific analysis of the HS method by Geem et al. (2001): '*A Rigorous Analysis of the Harmony Search Algorithm: How the Research Community can be Misled by a "Novel" Methodology*'. The paper appeared in the International Journal of Applied Meta-heuristic Computing. Weyland (2010) provides strong proof that the HS algorithm is no more than a special case of $(\mu + 1)$ Evolution Strategies method. An evolutionary meta-heuristic algorithm proposed by the German author Rechenberg (1973) nearly 30 years before.

Soerensen (2013a) reinforced this critical view by publishing: ‘Meta-heuristics - the metaphor exposed’ on the International Transactions in Operational Research. The author criticizes what he calls: ‘ . . . a true tsunami of “novel” meta-heuristic methods’.Soerensen (2013a: 2) states:

> ‘Since a few decades, every year has seen the publication of several papers claiming to present a "novel" method for optimization, based on a metaphor of a process that is often seemingly completely unrelated to optimization. The jumps of frogs, the refraction of light, the flowing of water to the sea, an orchestra playing, sperm cells moving to fertilize an egg, the spiraling movements of galaxies, the colonizing behavior of empires, the behavior of bats, birds, ants, bees, flies, and virtually every other species of insects - it seems that there is not a single natural or man-made process that cannot be used as a metaphor for yet another "novel" optimization method. Invariably, the authors of such papers promise that their "new" method is superior to methods previously published in the literature. ...such papers attract an impressive follow-up literature, in which a large number of optimization problems are subjected to the "novel" method, invariably with strikingly good results’. Soerensen (2013a: 11) concludes that:
>
> ‘..."novel" meta-heuristics based on new metaphors should be avoided if they cannot demonstrate a contribution to the field. To stress the point: renaming existing concepts does not count as a contribution’.

The author examines the justification for developing new algorithms pointing out fallacies and trying to target meta-heuristic research to more solid paths. The author shows that there is a lot of excellent research being undertaken in meta-heuristics, and research on the field is, nevertheless, progressing to ever more powerful methods.

2.4 Summary

This chapter has introduced the problem domain of dimensioning WDN and critically analyzed the body of knowledge in this problem field. The focus of the literature review was on representative methodological developments for identifying non-approached issues or gaps relevant for setting the objectives of this dissertation towards innovation in the dimensioning of branched WDN for irrigation (see next chapter).

The optimization problem of dimensioning a WDN, where discrete diameters are considered and where the hydraulic nonlinear constraints related to the energy conservation law (head losses in the pipe system) need to be considered explicitly, is very difficult to solve. In fact, this class of problems belongs to the MINLP problem type, which is formally called *NP-hard*. Research has approached these type of problems by making assumptions, wherever possible, to circumvent the known solution difficulties.

The methodology development started in the early years with LP applications for branched systems, without considering energy cost modeling and using fixed demand patterns and discharge flow distributions. These assumptions facilitated casting the problem as an LP. The only decision variables were the lengths of different pipe types available for each section (pipes with different diameters, roughness and unit head losses). This type of approach has several disadvantages, for example, irrigation management approaches are not easy to introduce without discrete variables (valves and irrigation shifts), consideration of trade-offs between diameters and energy costs are not easy or even impossible to track, and the approach does not consider systems on uneven terrains.

Applying LP to looped networks was not practicable given the impossibility of fixing discharge flow directions. The methodology evolved to Linear Programming Gradient approaches (LPG) considering multiple flow regimes in the loops satisfying given network demands at the nodes. The problem with these LPG approaches was the non-convexity of the objective function. The quality of the solution could not be assessed (local optima). Improvements appeared considering the minimization of non-convex objective functions that managed to assess the quality of the calculated solutions. Nonlinear Programming Gradient and other Nonlinear Programming methods tried to introduce the consideration of nonlinear terms e.g. energy costs in the objective function. Nevertheless, nonlinear algorithms cannot guarantee to find the global optimum of the problem. Furthermore, nonlinear methods are based on continuous diameters that have to be rounded to commercial available pipe diameters. This is a major disadvantage, not only because the methods do not guarantee global optimality, but also the conversion to discrete diameters will certainly not be optimal, and can even produce unfeasible solutions.

There is almost no research addressing the question raised by Walski, (2001) regarding the "Wrong Paradigm" of the overwhelming amount of research considering only minimizing the networks pipeline costs. For example, there is practically no research embedded in a Cost-Benefit framework for dimensioning WDNs.

The optimal layout of the network also became a concern and was introduced in methodologies and applications, where different approaches of the Steiner Tree/Forest method have been used to derive the shortest path layout of the networks. Given an optimal layout of the network the next interesting question is the optimal extension of the network. This question is hardly addressed in the literature, because dimensioning WDNs is not made in an investment or project appraisal framework. The majority of studies do not consider the inclusion of benefits of water distribution and only considers minimization of the networks' initial investment costs in the pipeline structure.

Most research is applied to gravity systems, the problems of nonlinearity of energy costs in the objective function of the optimization problem is sidestepped, the complexities of the trade-off modeling between energy and initial pipeline investment costs are avoided.

The majority of studies for dimensioning WDN for irrigation considers only fixed demand patterns and flow regimes. When considering only one water demand pattern there is no guarantee that the system selected is the least costly and fulfills the performance required. This is especially the case for on-demand irrigation systems. The distribution of flows through each section can change strongly during the year and across the network. These limitations have only recently been addressed. Only a few studies have included the analysis of water demand patterns and hydrant characteristic curves in the dimensioning methodology.

Most of the work for irrigation WDN is concentrated within on-demand irrigation schemes, only a few studies have analyzed the dimensioning of irrigation WDN with rotation schemes of irrigation. These studies have shown that partitioning the irrigation network in irrigation sub-units and different irrigation shifts can have a major impact on the overall cost of the system.

A shift from the classical closed mathematical programming models can be witnessed in recent years towards modeling approaches that circumvent the hydraulic nonlinear constraints by using external hydraulic simulation models. The majority of these approaches are based on Meta-heuristic Optimization Methods.

The Meta-heuristic Optimization methods are not able to guarantee a global optima, but can nevertheless achieve very good solutions. The problem with these more recent methods (besides the danger of being "caught" in local optima) rests on the exhaustive parameter setting necessary for modeling every different application. Research on meta-heuristics for irrigation WDN is relatively sparse, nevertheless, several difficulties have been reported in modeling branched irrigation networks, e.g. solutions inconsistencies between upstream and downstream selected diameters of the branches' pipelines.

There is a huge trend of research continuously developing new meta-heuristics, drawing on some metaphors from Nature. All these new algorithms are first tested on simple benchmark WDN. There are not many studies applying these new procedures to large real-world case studies. The difficulties in parameter setting and the search for global optima can be relatively easy to handle in the common small dimension benchmarks and so the performance of all of these metaphors is not completely tested and understood. Furthermore, the large majority of the meta-heuristic applications are for gravity WDN systems for domestic and industrial purposes. There are very few meta-heuristic studies considering pumped irrigation systems, where the trade-offs between initial investment costs in the pipeline structure and the life-cycle energy costs are addressed.

In the next chapter the objectives of this dissertation will target these unanswered questions and propose a new insight into a closed mathematical programming approach (MILP) for solving the problem of dimensioning a branched WDN for irrigation.

Chapter 3

Objectives

3.1 Introduction

The main goal of this dissertation is to close the methodological gap between WDN designs based only on minimization of the networks' costs and the necessity for more inclusive approaches based on cost-benefit analysis (CBA). A CBA optimization framework should be developed balancing the life-cycle benefits, life-cycle costs and initial investments of the WDN project. The optimization model developed should deliver a full economic analysis, optimal network size, pipeline diameters, pump system capacity (discharges, heads and power demanded per shift), as well as the optimal spatial distribution of simultaneous irrigating hydrants per shift.

To achieve this goal this dissertation will provide a new insight in approximating the MINLP mathematical nature of the original WDN problem developing a Mixed Integer Linear Programming model (MILP).

MILP models are most suitable for dealing with the discrete nature of the pipeline diameters in dimensioning a WDN. The development of MILP software tools has been impressive in the last few years, showing a continuous increase in the efficiency of optimization solvers for MILP models. This fact increasingly encourages the use of closed mathematical programming models to address these MINLP problems. Efficient techniques for piecewise approximations to non-linearities are available more than previously and are being increasingly applied (Padberg, 2000; Dambrosio, 2009, 2010). The MILP model in focus has an applied character and will include major complexities that have been neglected or only partially addressed by research. The model will make an original contribution to the body of knowledge on modeling-branched WDN for irrigation.

3.2 Specific objectives

The model targets explicitly identified research gaps that were evident in the revision of the current state of knowledge on dimensioning WDN.

The specific objectives of the model are:

1. *To cast the optimization model for dimensioning WDN in a cost-benefit analysis (CBA) framework.*

   The objective function of the WDN model should not simply minimize the costs of the WDN, but more importantly maximize the net-benefits of the investment over a suitable life-cycle. By including the benefits of each irrigation hydrant, it is possible to assess the efficiency of the investment by balancing irrigation sectors benefits, initial investment costs and energy recurrent costs for the life-cycle of the project. The objective function will basically be constructed as a Net Present Value (NPV), balancing the initial investment costs of the pipeline structure and pump system against the discounted recurrent irrigation benefits and recurrent costs of the irrigation network.

   Innovation 1: This dissertation responds in this way to a major criticism of WDN research: systematically negleting the inclusion of WDN benefits in the objective function of the proposed optimization models (Walski, 2001).

2. *To determine the optimal network size endogenously in the optimization process*

   This is a consequence of the first objective. By including the benefits of water distribution, it is possible to determine the optimal size of the irrigation network.

   Innovation 2: To the author's knowledge, no other study has followed this objective in such an inclusive model as the one presented here. More irrigation hydrants will only be included in the optimal solution, if their marginal benefit contribution to the objective function (CBA function) is at least as large as their marginal costs. Researchers have used different approaches of the Steiner Tree/Forest method for deriving the shortest path layout of the networks (Takahashi and Matsuyama, 1980). The departure size of the network is, nevertheless, always given. If the purpose is to implement a new WDN e.g. a new irrigation settlement such as the case-study of this dissertation, one of the most interesting questions is the optimal size of the network. This problem is economically very important in irrigation networks located on non-flat terrains, where new hydrants are connected sequentially towards unfavorable higher elevation directions (see case-study). The net-benefits of each hydrant should be compared to the costs of its connection and operation. This will determine the optimal size of the irrigation areas or farm-settlements.

3. *To determine endogenously the number and spatial distribution of nodes irrigating simultaneously in each shift.*

   The branched WDN considered in this dissertation should be operated on a rotational water delivery management system. Irrigation is to be scheduled considering plants' water needs, soil water-holding capacities and allowable irrigation intervals.

   Innovation 4: The shift pattern, i.e. determination of the number and spatial distribution of operating hydrants in the different irrigation shifts, is performed endogenously. This will have a great influence on the network's dimensioning. In this case, discharges in the different pipelines cannot be assumed *a priori*, and are endogenous in the optimization. Modelling the schedule is complex, because it will be optimized simultaneously with the size of the network using binary variables representing irrigation valves (closed=0, open=1). To the author's knowledge, this aspect has never been modelled in such an integrated way by a closed optimization model. The pump discharges and heads are also decision variables of the problem and can vary from shift to shift. This degree of freedom allows an odd number of hydrants to irrigate in each shift, which is an innovation compared with past research, where the pump's design discharge and head are normally fixed *a priori*.

4. *To track the impacts of different discounting rates, real diesel price developments and irrigation benefits changes on the optimal design of the network.*

   Different assumptions for the discounting rate, or for the development of real diesel prices and benefits, will interact and induce different designs of the network as a consequence of the trade-offs involved.

   Innovation 3: Modeling these trade-offs in a more complete analysis that can be found in the literature.

5. *Provide a user friendly visualization of the optimal network result using a geographical information system.*

   The model will communicate with a Geographical Information System (GIS) for spatial representation of the network size. A soft-linkage with the optimization language GAMS (Brooke et al., 2005) and ArcGIS was programmed in the open-source language PYTHON for this purpose (www.python.org).

The work is structured as follows:

Part I *introduces the problem domain of this dissertation, the current state of knowledge, unaddressed key issues and the objectives of this work.*

The following Chapter 4 introduces the case-study of this dissertation for applying the MILP approach developed. The case-study is an irrigation settlement located in Upper Egypt, in the Governorate of Aswan, which is part of Egypt's government desert land reclamation program for the Lake Nasser region.

Part II *presents the parameter estimates for the model.*

In Chapter 5 the hydraulic foundations and hydraulic parameters of the model are presented and estimated. In Chapter 6 irrigation management principles are introduced and the relevant parameters for irrigation management are calculated. In Chapter 7 the cost-benefit analysis (CBA) framework of the model is introduced. This chapter describes the objective function cast as the net-present value of the investment. The benefits generated by irrigation in each network hydrant, as well as the initial investment costs (pipelines and pump system), the operation and maintenance costs (OM), as well as the network's depreciation, are discussed in detail together with the discounting process adopted.

Part III *represents the main contribution of the present dissertation: the proposed MILP model is developed and operationalized.*

Chapter 8 introduces the original MINLP mathematical formulation of the model.

Chapter 9 presents the developed MILP model for approximating the MINLP formulation. The MILP model is operationalized through piecewise linear approximations of the non-linear model equations.

Chapter 10 introduces the methodology followed for solving MILP optimization problems.

Part IV *presents the results and conclusions of the work.*

Chapter 11 introduces the internal rate of return (IRR) as economic criteria for the selection of different WDN designs in the NPV framework. The chapter also discusses the expected effects on the NPV and WDN designs of the interactions between IRR, the escalating rate of diesel prices, developments in project benefits and project life-time. Chapter 12 presents the WDN design results of the different parameterizations. Chapter 13 presents the conclusion of the work.

Chapter 4

Case-study: Rural settlements in Upper Egypt

4.1 Introduction

The case-study of this dissertation is located in Upper Egypt, in the Governorate of Aswan (see the governorate localization in Figure 4-1). The developed optimization model is applied to dimension the WDN of the Kalabsha agricultural settlement located to the south of Aswan city and the Aswan High Dam (AHD) on the shores of Lake Nasser (see Figure 4-2). The Kalabsha settlement is included in the Egyptian government's desert land reclamation program for the Lake Nasser region. Lake Nasser is the largest artificial lake in the world and was created by the construction of the AHD between 1958 and 1970. The water impoundment began in 1964, achieving a maximum level of 183 masl, and forming a reservoir extending 292 km within the Egyptian border, where it is called Lake Nasser and 204 km within the Sudanese border, where it is called Lake Nubia (Osman, 1999; UNDP, 2002).

The Ministry of Agriculture and Land Reclamation (MALR) in cooperation with the World Food Program (WFP) started in 1989 with the implementation of smallholder settlements on the shores of Lake Nasser. The potential desert land reclamation on the Lake

Figure 4-1: The Aswan Governorate

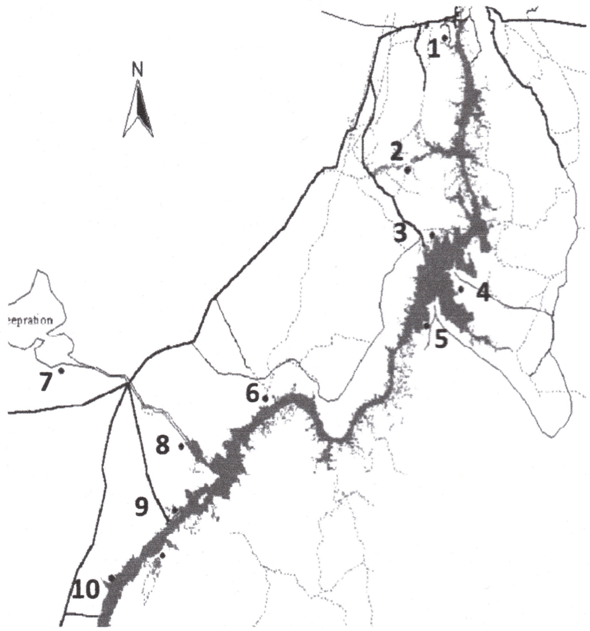
Source: USAID (2013)

Nasser shores was estimated at ca. 59,892 ha[1] (UNDP, 2002). Figure 4-2 shows the settlements and respective areas to be developed across this region to the south of the AHD. The settlements of Kalabsha, Garf Hussein and Tomas and Afia, were completed in the first phase of the development plan between 2005 and 2007. The Kalabsha settlement and the respective flood irrigation system was implemented with a total of 150 households and a total population of 750 people. Each farm-household is endowed with ca. 2 ha land, achieving in the first phase a total area of 300 ha. Water is pumped from a floating pump-

[1] Ca. 142,600 feddan (1 feddan = 0.42 hectares)

Figure 4-2: Lake Nasser Settlements

	Settlement	Area (feddan)
1.	Karkar	24,000
2.	Kalabsha	19,000
3.	Garf Hussein	18,300
4.	Allaqi North	16,000
5.	Allaqi South	
6.	Tomas and Afia	12,200
7.	Toshka depression	14,600
8.	Toshka	
9.	Abu Simbel	30,500
10.	Quastal	8,000
	TOTAL	142,600



Source: UNDP (2002)

ing station located in a main canal diverting water from Lake Nasser (the pumping station is located at node Nr. 1 in Figure 4-3). The floating station accommodates the fluctuations in the Lake Nasser water level, ranging from 176 masl in the month of July to 181 masl in December (UNDP, 2002). The irrigated plots were implemented between the contour lines of +181 masl and +189 masl altitude. Even though the installation of water-saving irrigation systems suitable for sandy soils is foreseen, the systems have still not been installed and flood irrigation is the practised method. Water is pumped from the floating pumping station into main distribution pipes delivering water to secondary earth canals.

4.2 The water distribution network: A study layout

The objective of the MALR is to increase, as much as possible, the current irrigation settlement area of 300 ha for 150 farm -households, which is currently still operated under a flood irrigation system. Investments should be made in a suitable WDN and on pressurized irrigation systems at the farm level. Based on MALR expert analysis about the quality of the soil and topography of the chosen location a total departing area of 500 ha for the network layout of this study was assumed. The disposition of the foreseen 25 hydrants in the network can be seen in Figure 4-3[2]. Each of these hydrants supplies 10 farms, a total irrigation area of 20 ha per hydrant. The network is considered to be divided into four main distribution lines $L_1 = (1, 2)$, $L_2 = (2, 7)$, $L_3 = (7, 15)$, $L_4 = (15, 23)$ and seven irrigation branches, i.e. branch I to branch VII in Figure 4-3.

Each irrigation branch contains its own set of pipeline sections (i, j), where j is always the irrigation hydrant of that section:

$$I = \{(2,3)\,,\ (3,4)\,,\ (4,5)\,,\ (5,6)\},$$

$$II = \{(7,8)\,,\ (8,9)\,,\ (9,10)\,,\ (10,11)\}\,,$$

$$III = \{(7,12)\,,\ (12,13)\,,\ (13,14)\},$$

$$IV = \{(15,16)\,;(16,17)\,;(17,18)\,;(18,19)\}\,,$$

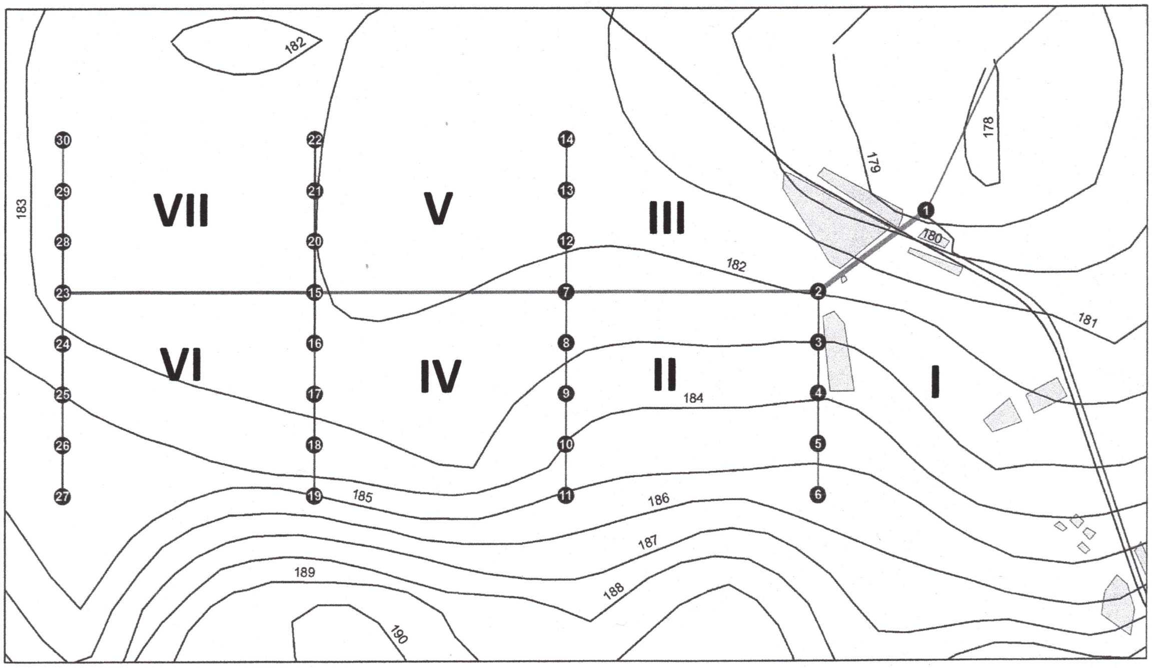
$$V = \{(15,20)\,,\ (20,21)\,,\ (21,22)\},$$

$$VI = \{(23,24)\,,\ (24,25)\,,\ (25,26)\,,\ (26,27)\},$$

$$VII = \{(23,28)\,,\ (28,29)\,,\ (29,30)\}\,.$$

[2] This is only a study-layout. There were no considerations taken in this work regarding the optimal paths connecting the hydrants in the network with the pumping system. These type of considerations should be done *a priori* determining the departing layout, i.e. before the optimization of the system. It normally involves minimal length Steiner forest methods (see Takahashi and Matsuyama, 1980). The application of the proposed model of this dissertation is nevertheless straight forward to any layout.

Figure 4-3: Proposed Water Distribution Network for Kalabsha



The lenghts of the main distribution lines are $L_1 = 533$ m and $L_2 = L_3 = L_4 = 1000$ m. The pipelines connecting the different hydrants in the network branches are all the same size, 200 m. The elevations of the different network nodes can be seen in the following Table 4.1

Table 4.1: Elevations in meters above sea level (m.a.s.l) of the different network nodes

z1 = 178.9	z7 = 182.2	z13 = 182.0	z19 = 185.0	z25 = 183.9
z2 = 181.9	z8 = 182.8	z14 = 182.0	z20 = 182.2	z26 = 184.3
z3 = 183.0	z9 = 183.4	z15 = 182.2	z21 = 182.2	z27 = 184.5
z4 = 183.9	z10 = 184.0	z16 = 182.2	z22 = 182.2	z28 = 182.7
z5 = 184.6	z11 = 185.1	z17 = 182.7	z23 = 182.7	z29 = 182.7
z6 = 185.4	z12 = 182.0	z18 = 183.3	z24 = 183.1	z30 = 185.0

Part II

Parameter Estimates

Chapter 5

Hydraulic foundations

5.1 Introduction

The objective of this chapter is to introduce the hydraulic problems involved when dimensioning a water distribution network (WDN). The WDN should deliver water to the irrigation hydrants (and farms) at a given operating pressure, demanding energy from the pumping system. The water in the pipelines of the network is subjected to different forces, and energy transformations occur when pumped water flows from one section to another. Energy is lost from the WDN system because of water friction against the pipeline's walls. The friction losses depend on the pipeline length and diameters, on the discharge and on the pipeline's material roughness. Higher diameters will produce lower friction losses and lower inherent recurrent energy costs from pumping, but are more expensive and demand higher initial investment costs. The optimal dimensioning of the WDN will balance these trade-offs for the life-cycle of the investment.

The remainder of this chapter introduces in Section 5.2 the concepts of the hydraulic energy balance necessary to understand the power needs when pumping water between two points of a water distribution network. Section 5.3 addresses the energy losses in the system, i.e. energy contained in water when flowing, that can be lost from the system through pipe friction and through obstructions to flow e.g. pipe bends, etc. This implies that more energy (pump power) needs to be delivered to the system to compensate for these losses. The chapter closes with Section 5.4 discussing the characteristic demand curve of the system, necessary pumping power and consequent energy costs.

5.2 Energy analysis

The proposed model for dimensioning the WDN must assure that for each node of the network the sum of inflows equals the sum of outflows from that node, i.e. the law of mass conservation in hydraulics. The model must further confirm that the system's pump provides enough pressure to overcome the height differences and friction losses between every pipeline section in the network (law of energy conservation).

Deriving the model's constraints obeying the law of mass conservation

According to this conservation law, the mass of water flowing through any section area A (m^2) of the pipeline network must be constant. This means, the fluid mass flow entering a control volume $A_1 \cdot \dfrac{ds_1}{dt}$ in $\mathrm{m}^3 \cdot \mathrm{s}^{-1}$, must be the same as the mass flow leaving the volume $A_2 \cdot \dfrac{ds_2}{dt}$ in $\mathrm{m}^3 \cdot \mathrm{s}^{-1}$, where A_1 and A_2 are the pipeline sections and ds_1 and ds_2 the incremental distances displaced in the incremental time interval dt, i.e.:

$$\rho A_1 \cdot \frac{ds_1}{dt} = \rho A_1 V_1 = \rho A_2 \cdot \frac{ds_2}{dt} = \rho A_2 V_2 = \rho Q \tag{5.1}$$

where, ρ ($\mathrm{kg} \cdot \mathrm{m}^{-3}$) is the specific mass, $\dfrac{ds}{dt}$ ($\mathrm{m} \cdot \mathrm{s}^{-1}$) is the velocity of the fluid, and $\rho Q = \rho A V$ ($\mathrm{m}^3 \cdot \mathrm{s}^{-1}$) the called the mass discharge or mass flow rate. This means that, if the cross-section area of the pipe narrows somewhere along the flow circuit, the velocity of flow must increase in this section and vice-versa (Hwang et al., 1996; Bloomer, 2000; White, 2003).

The law of mass conservation is implemented for the branched system in each node as:

$$Q_{ij}^{IN} - \sum_{\overline{ij}} Q_{\overline{ij}}^{OUT} = 0;\ \forall ij \text{ and } \overline{ij}^{1}$$

Deriving model's constraint preserving the energy conservation law

Forces acting on a fluid are said to produce *Work*[2] if the fluid is set in motion, i.e. the net force acting induces a velocity change, a change in the kinetic energy of the fluid body, i.e:

$$dW = F \cdot ds = FV \cdot dt = F \cdot \frac{ds}{dt} dt\ , \tag{5.2}$$

where, F is a vector of forces acting on the fluid, V is the vector of velocities and dt the incremental time interval over which the force acts (Bloomer, 2000; White, 2003; Serway et al., 2004).

The work done by the pressure forces on pipe Sections 1 and 2 of cross-section areas A_1 and A_2 in time increment dt is the product of the respective pressure force $F = PA$, with the incremental mass displacement ds (Hwang et al., 1996; Bloomer, 2000; White, 2003).

For section 1,

$$P_1 A_1 \cdot ds_1 = P_1 A_1 \left(V_1 \cdot dt\right) = P_1 \left(A_1 V_1\right) \cdot dt = P_1 Q \cdot dt \tag{5.3}$$

[1] The set $\overline{ij}$ contains all pipeline sections outflowing from the respective node.

[2] *Work* (W) *is normally given in standard SI units like Joule* (J)*. A Joule is the Work done by a force of one Newton acting over a distance of one meter. The SI unit Newton expresses a force and it is better understood in connection with the Newton's second law of motion, which states that one newton applied to a mass of 1 kg will set this mass in motion with an acceleration of 1* $m \cdot s^{-2}$*; i.e. ´*$1\ N = 1\ kg \cdot \left(m \cdot s^{-2}\right)$*, (Bloomer, 2000; White, 2003; Serway et al., 2004)*

For section 2[3],

$$- P_2 A_2 \cdot ds_2 = -P_2 A_2 \left(V_2 \cdot dt\right) = -P_2 \left(A_2 V_2\right) \cdot dt = P_2 Q \cdot dt \tag{5.4}$$

The work done by the gravity force on moving the mass of water from section 1, a geographical point with elevation z_1, to section 2 with elevation z_2, is given by the gravity force multiplied by the vertical distance $\left(z_1 - z_2\right)$. The gravity force is defined as the water specific weight $\gamma = \rho g$, multiplied by the cross-section area A_1. If we consider again the induced incremental mass displacement ds_1, i.e. $\rho g A_1 \cdot ds_1 = \rho g \left(A_1 V_1\right) \cdot dt$. The work done by the gravity force in time increment dt is:

$$\left(\rho g \left(A_1 V_1\right) \cdot dt\right) \cdot \left(z_1 - z_2\right) = \left(\rho g Q \cdot dt\right) \cdot \left(z_1 - z_2\right) \tag{5.5}$$

The total work done on the flowing water mass is expressed as the change in the kinetic energy of the fluid as mentioned above, and can be mathematically explained by the Law of Momentum Conservation, i.e. the sum of forces[4] $\sum \overrightarrow{F}$ acting on a fluid, will induce acceleration in the fluid according to Newton's second law, which is to say,

$$\sum \overrightarrow{F} = m\overrightarrow{a} = m\frac{d\overrightarrow{V}}{dt} = \frac{d}{dt}\left(m\overrightarrow{V}\right) = m\frac{\overrightarrow{V}_2 - \overrightarrow{V}_1}{\Delta t}, \tag{5.6}$$

where, m is the mass of fluid, and a the acceleration. The term $\frac{m}{\Delta t}$ equals the mass flow rate ρQ, (see Hwang et al., 1996). Substituting in Equation (5.6) gives,

$$\sum \overrightarrow{F} = \rho Q \left(\overrightarrow{V}_2 - \overrightarrow{V}_1\right) \tag{5.7}$$

See Hwang et al., (1996); Bloomer, (2000) and White, (2003) for more details.

[3] The negative sign means the force is done in the opposite direction to the motion of the fluid. It means the force acting at the other end of the pipeline.

[4] $\sum \overrightarrow{F}$ is an expression for vector quantities, the sum of the forces in the different three dimension directions $\sum F_x$, $\sum F_y$, and $\sum F_z$ (see Hwang et al., 1996 for more details).

It can be shown by integrating (5.7) in t, the total work done on the fluid mass by all the forces, i.e. $\sum \overrightarrow{F}$, is equal to the total change in *kinetic energy:*

$$\frac{1}{2}mV_2^2 - \frac{1}{2}mV_1^2 = \left(\frac{1}{2}\rho Q \cdot dt\right) \cdot \left(V_2^2 - V_1^2\right) \ , \tag{5.8}$$

(see Hwang et al., 1996; Bloomer, 2000; White, 2003).

Setting now the sum of the work done by pressure and gravity forces equal to the changes in kinetic energy gives:

$$P_1 Q \cdot dt - P_2 Q \cdot dt + (\rho g Q \cdot dt) \cdot (z_1 - z_2) = \left(\frac{1}{2}\rho Q \cdot dt\right) \cdot \left(V_2^2 - V_1^2\right) \ . \tag{5.9}$$

Dividing by $\rho g Q \cdot dt$,

$$\frac{P_1}{\rho g} - \frac{P_2}{\rho g} + (z_1 - z_2) = \frac{1}{2g}V_2^2 - \frac{1}{2g}V_1^2. \tag{5.10}$$

Reshaping last equation gives the known Bernoulli equation in energy per unit weight of water,

$$\frac{P_1}{\rho g} + \frac{V_1^2}{2g} + z_1 = \frac{P_2}{\rho g} + \frac{V_2^2}{2g} + z_2 \tag{5.11}$$

(See Hwang et al., 1996; Bloomer, 2000; White, 2003 and Trifunovic, 2006).

The Bernoulli equation give us the energy equivalence between the two sections of the pipeline that has to be dimensioned. The first term on the left side of Equation (5.11) is called the *pressure head*, the second term the *velocity head*, and the third the *elevation head*. These three terms account for a large portion of all the energy forms contained in a fluid flowing through the pipes sections 1 and 2. During water flow in the system a part of the energy contained in the fluid can be transformed to other forms that do not produce any $Work$ in the system of study. These types of energy losses for fluid flow in closed pipelines commonly come from friction of the fluid with the pipe walls and obstructions to flow, such as pipe bends, valves and other network components. To account for these energy losses from the system, the energy conservation law is applied in fluid mechanics (Hwang et al., 1996; Bloomer, 2000; White, 2003).

The energy conservation law of fluid mechanics can be formulated as,

$$\frac{P_1}{\rho g} + \frac{V_1^2}{2g} + z_1 = \frac{P_2}{\rho g} + \frac{V_2^2}{2g} + z_2 \pm \Delta E \tag{5.12}$$

where ΔE is the energy transformed in the system.

$$E_1 = E_2 \pm \Delta E, \tag{5.13}$$

If there is energy dissipation, the sum of the pressure and velocity heads in Section 2 is lower than in Section 1 and ΔE is positive. This means that energy is lost out of the system due to the fluid's friction on pipe's walls and obstructions to flow.

The component for head losses due to friction and obstructions to flow can be expressed as:

$$\Delta E = h_f + h_m \tag{5.14}$$

where, the term h_f represents the so-called *major losses* occurring due to friction and viscous dissipation in flowing water. The term h_m represents the head *minor losses*, mainly in pipe's curves, bends, or valves, i.e. all the components in the path of water that restrain flow.

The elevation, pressure and velocity heads are measured in meters of water column (mwc) and given in relation to a reference level, e.g. meters above sea level, (Hwang et al., 1996; Bloomer, 2000; White, 2003; Trifunovic, 2006).

5.3 Energy losses from the system

As mentioned in the last section, the energy losses are caused by friction between water and the pipe walls, and also by turbulences caused by obstacles to flow, e.g. valves, bends, etc. The losses are a function of the flow in the pipe and can be expressed by,

$$\Delta E = h_f + h_m = R_f \cdot Q^{n_f} + R_m \cdot Q^{n_m} \tag{5.15}$$

where, R_f represents the resistance to flow due to pipeline friction and R_m represents minor losses due to obstructions to flow (valves, pipe bends, etc.)[5]. Q is the discharge in the respective pipeline. For estimating the resistance coefficients the most popular formulations are the Darcy-Weisbach and the Hazen-Williams Equations (Hwang et al., 1996; Trifunovic, 2006). Both equations describe the resistance to flow in a pipe by a functional form that is basically dependent on the length of the pipeline, the diameter, and roughness of the pipe (Hwang et al., 1996; Bloomer, 2000; White, 2003; Trifunovic, 2006; Prabhata et al., 2008).

The Darcy-Weisbach Equation

Giving a pipeline with length L (m) and diameter D (m), the Darcy-Weisbach equation (DW) for the resistance term in (5.15) is given as,

$$R_f = \frac{8\lambda L}{\pi^2 g D^5} \tag{5.16}$$

where g is gravity's acceleration (9.81 $\text{m} \cdot \text{s}^{-2}$) and the coefficient λ is a friction factor related to the roughness of the pipe.

[5] In this work it is assumed that these losses are small enough when compared with the pipeline friction losses and R_m will be neglected from now on.

The λ coefficient is calculated through the Colebrook-White Equation as:

$$\frac{1}{\sqrt{\lambda}} = -2\log\left[\frac{2.51}{\mathrm{Re}\sqrt{\lambda}} + \frac{k}{3.7D}\right] \tag{5.17}$$

where the coefficient k is the pipe roughness in mm, D is again the pipe's inside diameter in mm and the term Re represents the *Reynolds* number. The *Reynolds* number describes the flow regime of the system, establishing a relationship between velocity of flow V ($\mathrm{m \cdot s^{-1}}$), the diameter of the pipe involved and the kinematic viscosity of water υ ($\mathrm{m^2 \cdot s^{-1}}$).

$$\mathrm{Re} = \frac{V \cdot D}{\upsilon} \tag{5.18}$$

For the purpose of the analysis, laminar and turbulent flows can be distinguished. Energy losses can be divided into friction against the pipe walls and energy dissipation in the fluid due to its viscosity. The turbulent flow has a higher velocity gradient and friction in the contact with the pipe walls as the laminar flow regime. This implies higher energy losses in the case of the turbulent flow, i.e. a higher Reynolds number (see Equation 5.18). Higher turbulence also induces more energy dissipation due to viscosity forces. Higher velocities are characteristic of turbulent flows and higher Reynolds numbers. The flow regime is characterized as laminar, if $Re < 2000$, in transition if $Re \approx 2000 - 4000$ and turbulent, if $Re > 4000$ (Hwang et al., 1996; Bloomer, 2000; White, 2003; Trifunovic, 2006).

For security and reliability reasons the velocity regime in irrigation systems is advised to be held between 1.5 and 3.0 $\mathrm{m \cdot s^{-1}}$ (Walski, 2001). This fact implies that turbulent flows are the rule in water distribution networks for the standard pipe diameters and temperatures involved. After determining the Re number, the λ coefficient can be calculated from Colebrook equation in 5.17, (Colebrook, 1939), or more often from its graphical representation, the so-called *Moody*-diagram (Hwang et al., 1996; Bloomer, 2000; White, 2003; Trifunovic, 2006; Prabhata et al., 2008). The DW formulation sets $n_f = 2$ in (5.15) giving a friction loss h_f of,

$$h_f = \frac{8\lambda L}{\pi^2 g D^5} \cdot Q^2 \tag{5.19}$$

where Q is given in $\mathrm{m}^3 \cdot \mathrm{s}^{-1}$(Hwang et al., 1996; Bloomer, 2000; White, 2003; Trifunovic, 2006; Prabhata et al., 2008).

The Hazen-Williams Equation

The Hazen-Williams (HW) Equation is an empirically derived formula very much used in practice due to its simplicity (Hwang et al., 1996; Trifunovic, 2006; Prabhata et al., 2008). The formula considers an empirical estimated roughness coefficient C_{hw}, given for different material types, not needing any calculations of the Re number or usage of the Moody diagram to estimate λ. This fact makes the HW formula more simple to program and implement, when compared with the DW equation. Moreover, although the DW equation is considered more accurate, the calculations of the HW equation are quite precise for medium and large diameters and still widely used in engineering applications (Trifunovic, 2006). The equation is given as:

$$R_f = \frac{10.68 L}{C_{hw}^{1.852} D^{4.87}} \tag{5.20}$$

The HW formulation sets $n_f = 1.852$ in (5.15) giving a friction loss h_f of,

$$h_f = \frac{10.68 L}{C_{hw}^{1.852} D^{4.87}} \cdot Q^{1.852} \tag{5.21}$$

The HW roughness coefficients for different materials are given in Table 5.1. The parameter 10.68 is a unit conversion factor for the case when L and D are in meters and the discharge Q is in $\mathrm{m}^3 \cdot \mathrm{s}^{-1}$ (Trifunovic, 2006).

As mentioned above, the HW equation is much more simple to program and was chosen for calculating the head-losses due to friction in the optimization model of this thesis.

Table 5.1: Hazen Williams roughness coefficients C_{HW} for different pipe materials and diameters

	(75mm)	(150mm)	(300mm)	(600mm)
Uncoated cast iron	121	125	130	132
Coated cast iron	129	133	138	140
Uncoated steel	142	145	147	150
Coated steel	137	142	145	148
Galvanized iron	129	133	—	—
PVC	142	145	147	150

Source: modified Trifunovic (2006)

Including it in Equation 5.12 and reformulating gives the energy conservation equation as:

$$\frac{P_1}{\rho g} + \frac{V_1^2}{2g} + z_1 = \frac{P_2}{\rho g} + \frac{V_2^2}{2g} + z_2 + \frac{10.68L}{C_{hw}^{1.852} D^{4.87}} \cdot Q^{1.852} \quad (5.22)$$

The elevation of the different sections of the network (z_i)

A topographical map of the irrigation network can be seen in Figure 4-3, Chapter 4. This map was produced in a Geographic Information System (GIS) based on a digital elevation model (DEM). The elevations of the different nodes (z_j) can be depicted from the contour lines and are presented in Table 5.2 below.

Table 5.2: Elevations in meters above sea level (m.a.s.l) of the different network nodes

z1 = 178.9	z11 = 185.1	z21 = 182.2
z2 = 181.9	z12 = 182.0	z22 = 182.2
z3 = 183.0	z13 = 182.0	z23 = 182.7
z4 = 183.9	z14 = 182.0	z24 = 183.1
z5 = 184.6	z15 = 182.2	z25 = 183.9
z6 = 185.4	z16 = 182.2	z26 = 184.3
z7 = 182.2	z17 = 182.7	z27 = 184.5
z8 = 182.8	z18 = 183.3	z28 = 182.7
z9 = 183.4	z19 = 185.0	z29 = 182.7
z10 = 184.0	z20 = 182.2	z30 = 185.0

5.4 Pumping power

The *water power* N_p necessary for the demanded pump discharge Q_{pu} and head P_{pu} is calculated as:

$$N_p = \rho g \cdot Q_{pu} \cdot P_{pu} \text{ [W]}$$

where again Q_{pu} is given in $\mathrm{m}^3 \cdot \mathrm{s}^{-1}$, and the head P_{pu} in m. The parameter ρ is the mass density of water, 1000 $\mathrm{kg} \cdot \mathrm{m}^{-3}$ at 20 $°C$, g is the acceleration due to gravity 9.81 $\mathrm{m} \cdot \mathrm{s}^{-2}$.

The power to operate the pump will be higher, due to energy losses in the pump and electric motor, one must convert the *water power*[6] produced by the pump into *electric power* used by the pump. The conversion is performed by using efficiency relationships (Walski, 2001; Trifunovic, 2006):

$$\eta_p = \frac{water\ power_{out}}{pump\ power_{in}}\ , \tag{5.23}$$

$$\eta_m = \frac{pump\ power_{out}}{electric\ power_{in}}\ , \tag{5.24}$$

where,

$$\begin{aligned} \eta_p &= \text{pump efficiency (\%)} \\ \eta_m &= \text{electric motor efficiency (\%)} \end{aligned}$$

The power required by the pump from the electric motor N_m is now given by:

$$N_m = \frac{\rho g \cdot Q_{pu} \cdot P_{pu}}{\eta_m \cdot \eta_p} \text{ [W]}$$

[6] *The name water power is used by Walski et al. (2001).*

The case-study of this work is located in an off-grid region and the electric motor driving the pump is supplied with electricity produced by diesel generators. In this case we have to do with one more source of energy losses, and another efficiency needs to be accounted for. It will be called the generator efficiency η_g.

$$\eta_g = \frac{electric\ power_{out}}{diesel\ power_{in}}\ (\%)\ .$$

The whole system's efficiency will be called in this work the *diesel-to-water efficiency* η_{d-w} (%) given by:

$$\eta_{d-w} = \eta_g \cdot \eta_m \cdot \eta_p \tag{5.25}$$

The *diesel-to-water efficiency is assumed in this work to be* $\eta_{d-w} = 40\%$, an optimistic value for off-grid systems[7]. The diesel power necessary to operate the pump is given by the expression:

$$PW = \frac{\rho g \cdot Q_{pu} \cdot P_{pu}}{\eta_{d-w}}\ [\mathrm{W}] \tag{5.26}$$

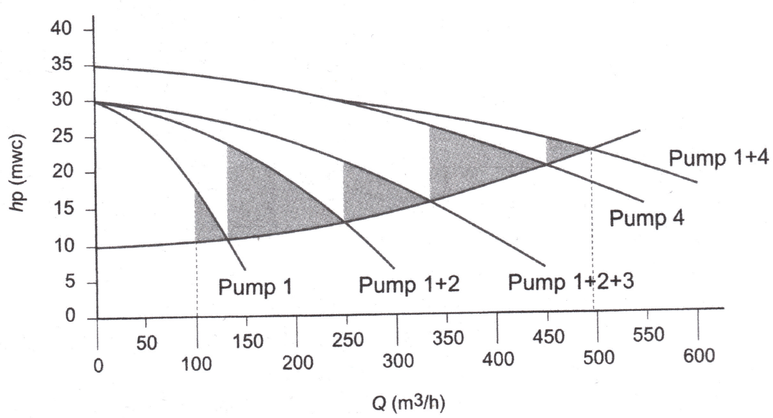
If only a single pump unit is moving water in the WDN there will always be a trade-off between the pump's supplied flow and head, i.e. the higher the water flow demanded by the network in one operation shift, the lower can be the pumping head provided (see Figure 5-2).

[7] This assumption is based on personal experience and expert discussions, the performance of generators, and electric motors available in Egyptian markets was not analysed.

The system's characteristic curve

In this work the optimization model determines the whole size of the network and the spatial distribution of hydrants simultaneously irrigating (shift pattern). The model further determines the discharge and head demanded from the pump system in each irrigation shift, i.e. the system's characteristic curve. It is assumed in this work that the model's calculated operating point for each shift (Q_{pu}, P_{pu}) can be assured by combining single pumps in parallel or in series. For a combination of pumps operating in parallel, a composite pump curve can be seen in Figure 5-1 (see Trifunovic, 2006 for more details)[8].

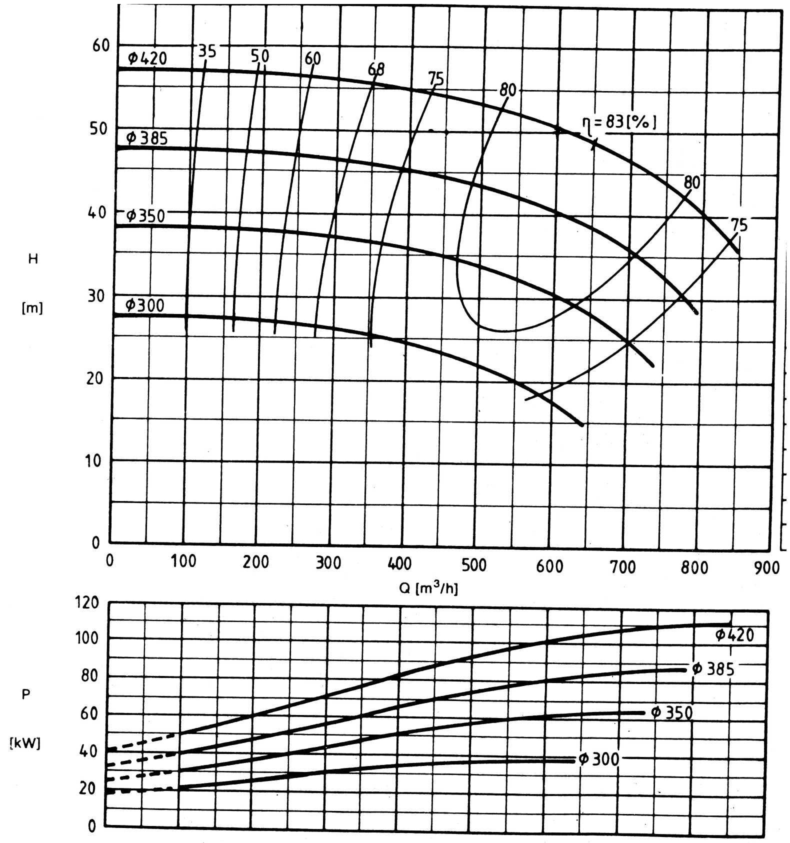
Figure 5-1: Pump sizes in parallel arrangement



Source: Trifunovic (2006)

[8] Pumps in a serial arrangement, supply the same discharge for an increased head (the opposite to a parallel connection)

Figure 5-2: Characteristic curves of a centrifugal pump



Source: Allweiler and Farid company

5.5 Summary

In this chapter two important model hydraulic equations were introduced. The first was the energy balance Equation 5.22, which represents the major non-linear hydraulic constraints of the optimization model. These constraints determine the energy balance in each section of the network accounting for head losses under every different flow regime and pipeline diameter:

$$\frac{P_i}{\rho g} + \frac{V_i^2}{2g} + z_i = \frac{P_j}{\rho g} + \frac{V_j^2}{2g} + z_j + \frac{10.68L}{C_{hw}^{1.852} D^{4.87}} \cdot Q^{1.852} \ .$$

As one can see, the constraint expresses clearly the trade-offs between the initial investment costs in the pipelines and recurrent energy costs. The HW component on the right side of the equation shows that higher diameters (D) will lower the head losses in the pipe section and consequently lower the energy demand from the system's pump (given the same discharge Q). Higher diameters for the pipeline section would reduce energy costs but imply higher initial investments. Solving for P_i in the above equation gives:

$$\frac{P_i}{\rho g} = (z_j - z_i) + \frac{10.68L}{C_{hw}^{1.852} D^{4.87}} \cdot Q^{1.852} + \frac{P_j}{\rho g} \ .$$

We can see that, the pressure head at the entrance of a pipeline in the network $\frac{P_i}{\rho g}$, must be enough to overcome the height differences between the section edges $(z_j - z_i)$, plus the pressure loss due to friction $\frac{10.68L}{C_{hw}^{1.852} D^{4.87}} \cdot Q^{1.852}$, and plus the minimum pressure head $\frac{P_j}{\rho g}$ demanded at the end of the pipeline or irrigation hydrant.

The other important derived equation is the system's power function. The model calculates the operating points of the network in the different shifts, i.e. the characteristic curve of the system. It is assumed that the power demanded at these operating points can be delivered by the system through a suitable combination of pumps in parallel or in series. The power demanded for calculating the pump system investment and recurrent operating costs was given as:

$$PW = \frac{\rho g \cdot Q_{pu} \cdot P_{pu}}{\eta_{d-w}} \tag{5.27}$$

Multiplying the power equation by the operational hours per year and by the energy cost rate will give the annual energy costs of the water distribution network. The operation, maintenance, and depreciation, as well as the initial investment costs for the pumping system will be dealt with in Chapter 7.

Chapter 6

Irrigation management

6.1 Introduction

The objective of this chapter is to introduce important irrigation concepts and management constraints that condition the design of the water distribution network. Important management parameters related to soil-water relationships and irrigation scheduling need to be understood and estimated for the optimization model. The irrigation management type selected for the WDN is essential for dimensioning. There are basically two types of water delivery systems to be considered when designing WDN for agriculture: (1) on-demand, and (2) rotational water delivery systems (Alandi et al., 2001; Burton, 2010). In on-demand systems the flow delivered by the pump is designed based on probabilistic principles. For example, the number of hydrants of the network operating simultaneously is a random variable. The dimensioning is not made for the sum of all hydrants discharges but only for the most probable maximal number of hydrants operating simultaneously during the peak water demand period (Clement, 1966; Clement and Galand, 1979; Clemmens and Bos, 1990). The irrigation settlement of the case-study of this dissertation should work on a rotational system of water delivery. On-demand systems are not suitable for this kind of settlement. The WDN for Kalabsha will serve an area of, at most, 500 ha. All farmers are faced with the same weather and soil conditions and the cropping patterns and sowing times are basically the same across all farms in the settlement. This means that farmers have the 'same' water demands at nearly the same time. An on-demand system would not be feasible, as the probability that all farmers irrigate simultaneously is too high for such conditions and the system would collapse. The preferred system for this situation is the rotational system.

In this type of system irrigation needs to be triggered before the water available for crops in the soil is lower than a certain suitable threshold. The settlement's soil has a specific capacity of holding water and, as will be seen, this is one of the main drivers for irrigation triggering, and for capacity dimensioning of the water distribution network. The higher the water-holding capacity of the soil (W_A), the longer plants can be held without irrigation, i.e. the interval between irrigation events can be made longer, and this can have management advantages. Determining the length of the irrigation interval determines the amount of water to be delivered to the field in one irrigation event. Frequent irrigations imply a lower discharge (capacity) of the system. Less frequent irrigations will demand higher discharges because it is the cumulated water consumed during the irrigation interval that needs to be replenished. The design of the water distribution system will have to take these factors into consideration, as they influence the initial investment costs in pipelines and pump, as well as the recurrent energy costs of operating the system.

The WDN *must be dimensioned to satisfy the peak water demand in the agricultural year*. It is the water demand during the peak period that determines the dimensions of the pipelines and sets the discharge capacity and design head of the system (Keller and Bliesner, 1990). If the system is able to deliver enough water for the peak water demand period, it should also be able to deliver enough water in the other crop growth stages. Irrigation in the peak water demand period needs to be concluded in a maximum allowed time interval before water is exhausted from the soil. The maximum allowed irrigation interval for the peak irrigation period $\left(F^{peak}\right)$ determines the dimensioning of the network.

The pumping costs depend on the total number of irrigation hours per year. Calculating the total operating hours of the system based on the peak water demand and on the maximum allowed irrigation interval for this peak period would overestimate the total operating hours and diesel costs of the network. To avoid this problem, the operating costs of the network are calculated based on average irrigation intervals for the winter $\left(F^W\right)$ and summer season $\left(F^S\right)$.

Conclusion: the WDN pipeline and pump dimensioning is based on the peak water demand in the year. The total operation hours of the WDN and the corresponding diesel costs, on the other hand, will not be estimated on peak values but on the average irrigation intervals in the two crop seasons.

The remainder of the Chapter starts by introducing in Section 6.2 the methods for calculating crop water demands given local meteorological data. Section 6.3 introduces the concept of the irrigation interval and derives the maximal and average lengths of the irrigation interval given the Kalabsha soil types. Section 6.4 calculates the maximum allowed number of irrigation shifts for the Kalabsha conditions and Section 6.5 closes by calculating the average total irrigation hours per year.

6.2 Crop water demand

The daily water requirement of a crop ($\mathrm{mm} \cdot \mathrm{d}^{-1}$) expresses the water needed for replenishing the daily rate of water lost through evaporation from the soil surface and transpiration of the crop. The composition of these two separate processes is called evapotranspiration (Allen et al., 1998; Goyal, 2012). Soil water evaporation is a thermo-dynamic process by which water is converted to vapor. The energy needed for the process is delivered by the solar radiation incidence on the surface under observation. Wind also has an important role in conducting the evaporation process. Wind removes recently evaporated water in the surroundings of the soil surface causing a "drying" effect. The parameters for calculating the evaporation are basically the solar radiation, temperature, air relative humidity and wind speed. The soil water content also plays a predominant role in evaporation and is influenced by rain and irrigation practices. If the soil water content is low, there will not be enough water to be transported to the surface for evaporation and the evaporation rate decreases (Allen et al., 1998; Goyal, 2012). Crop transpiration, on the other hand, is the process by which water contained in plants tissues is released as vapor to the surroundings. Transpiration also depends on the energy supply (solar radiation and temperature) and also on the differential of vapor pressures induced by wind speed. Another important factor is, again, the soil water content. Only the water available to the plant through the root system will be available for transpiration. The two processes of evaporation and transpiration occur simultaneously. Evaporation decreases though relative to transpiration during the growth process, because of the canopy development and increasing soil shading. When the canopy is fully developed, transpiration becomes the main water loss process (Allen et al., 1998; Goyal, 2012).

A direct method of calculating crop evapotranspiration consists of applying the FAO Penman-Monteith approach, which uses meteorological data, and many complex crop specific parameters and calculations[1]. Because comprehensive information on many crops is still missing and difficult to estimate, the Penman-Monteith Equation is normally only used for calculating crops' evapotranspiration through an indirect approach.

The Penman-Monteith indirect crop coefficient approach

In the first step evapotranspiration is calculated through the FAO Penman-Monteith Equation only for a standard **reference grass** under well defined reference characteristics[2], cal-

[1] The exposition of the Penmann-Monteith equation is not in the scope of this dissertation. Specific crop parameters are needed such as, the albedo and aerodynamic characteristics of the plants. The reader is refered to the FAO Irrigation and Drainage Paper No. 56 for more detailed information.

[2] According to (Allen et al., 1998: 15), 'The reference surface is a hypothetical grass reference crop with an assumed crop height of 0.12 m, a fixed surface resistance of 70 s m-1 and an albedo of 0.23. The reference surface closely resembles an extensive surface of green, well-watered grass of uniform height, actively growing and completely shading the ground. The fixed surface resistance of 70 s m-1 implies a moderately dry soil surface resulting from about a weekly irrigation frequency'.

culating what is called the Reference Evapotranspiration (ET_0)[3] for this reference grass. The Reference Evapotranspiration (ET_0) for the case-study region was calculated using the FAO CROPWAT 8.0 model, which applies the Penman-Monteith Equation to a standard grass under the local climatic conditions (FAO, 2013). In Table 6.1 the author's own calculations are presented for the case-study region of this work, using meteorological data from the FAOCLIM data base for the Kalabsha region in Upper Egypt[4]. The climatological station is located in the governorate of Aswan, at 194 *masl*, with latitude 23.97 $\deg .N$ and longitude 32.78 $\deg .E$. The data sets collected from the climatological station were maximal average temperatures, minimum average temperatures, relative humidity and solar radiation.

In a second step the calculated ET_0 is related empirically to the respective crop evapotranspiration, also called potential crop evapotranspiration (ET_p). The empirical relationship between the reference grass and a very large number of crops has been estimated and analyzed by several authors, in a vast number of agricultural experiments under different climatic conditions (Allen et al., 1998, Goyal, 2012). These empirical relationships are expressed by the so called crop coefficients θ_c. The crop coefficients equal the ratio ET_p/ET_0, i.e. the evapotranspiration of the specific crop in relation to the evapotranspiration of the reference grass according to the predefined conditions (Doorenbos and Pruitt 1977; Allen et al., 1998; Goyal, 2012). As Allen et al. (1998) states:

> 'The crop coefficient integrates the effect of characteristics that distinguish a typical field crop from the grass reference, which has a constant appearance and a complete ground cover. Consequently, different crops will have different θ_c coefficients'.

[3] According to Allen et al., 1998, "...the purpose is to exclude the effects of crop physiology, crop development and management practices. Water is available and soil conditions don't affect calculations. The evaporative demand of the surrounding atmosphere is "isolated". In this way a comparability of ET_0 is established for different locations, because they refer to the same reference surface and plant".

[4] http://geonetwork3.fao.org/climpag/agroclimdb_en.php

Table 6.1: Meteorological data for Kalabsha and ET0 calculation

	Max. Temp. ° C	Min. Temp. ° C	Humidity %	Wind speed km/h	Sunshine h	Solar radiation MJ/m^2/d	ET_0 mm/d	Day d	Monthly ET_0 mm/m
Month									
January	29.5	3.8	43	380.2	9.7	17.4	6.6	31	204.6
February	28.3	8.3	42	362.9	9.8	19.7	6.48	28	181.44
March	37.0	3.0	28	423.4	9.7	22.1	10.35	31	320.85
April	42.8	12.1	20	440.6	10.4	25	12.82	30	384.6
May	42.2	17.8	21	475.2	10.9	26.4	13.38	31	414.78
June	45.2	21.2	18	466.6	12.1	28.2	14.48	30	434.4
July	47.5	25.2	20	432	12.1	28.1	14.42	31	447.02
August	45	24	21	449.3	11.6	26.9	13.76	31	426.56
September	42.8	23	23	440.6	10.4	23.7	12.39	30	371.7
October	39	16.9	30	380.2	9.9	20.6	9.6	31	297.6
November	34	4	43	406.1	9.9	18.1	8.05	30	241.5
December	28.5	3	46	509.8	9.4	16.2	7.15	31	221.65

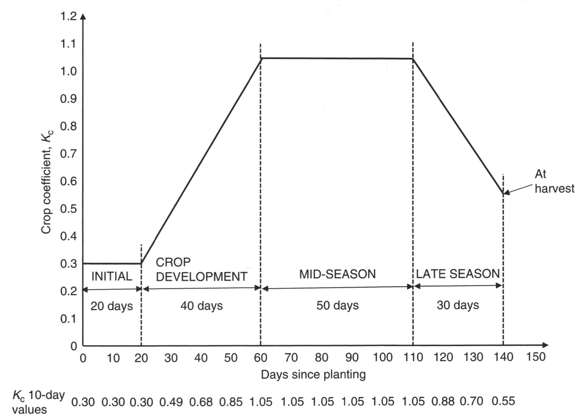
Source: Own calculations using CROPWAT 8.0

For a given climate, the crop's ET_p is set in relation to the reference grass evapotranspiration through the relation,

$$ET_p = \theta_c \cdot ET_0 \tag{6.1}$$

In figure 6-1 it can be observed that the crop coefficients θ_c will change according to the growth stage of the crop and its physiological development in four distinct growth stages: initial, crop development, mid-season (peak-season) and late season stage (Doorenbos and Pruitt 1977; Allen et al., 1998; Goyal, 2012). The specific ET_p for the local crop varieties was calculated on a daily basis for the specific growth periods by multiplying the daily reference evapotranspiration ET_0 by the respective crop coefficient value in the growth stage (see Table 6.2 for the monthly aggregated values)[5].

Figure 6-1: Crop coefficients development during the crop growth phases



Source: Burton (2010)

[5]In this dissertation, the climatic data used in the ET_0 calculations refer to the agro-climatic zone of Upper-Egypt, governorate of Aswan, and were available at the FAOCLIM data base. The required variables for calculation are, maximum and minimum temperatures (T in °C), mean relative humidity (RH *in* %), wind speed $\left(\text{km. d}^{-1}\right)$ and the mean actual sunshine duration $\left(\text{h. d}^{-1}\right)$.

Table 6.2: Crop coefficients off typical crops across growth stages

	θ_c^{ini}	θ_c^{peak}	θ_c^{end}
Vegetables (Solanaceae)			
Egg Plant	0.6	1.05	0.90
Sweet Peppers	0.6	1.05	0.90
Tomato	0.6	1.15	0.70-0.90
Vegetables, (Cucurbitaceae)			
Cucumber	0.6	1.00	0.75
Squash, Zucchini	0.5	0.95	0.75
Sweet Melons	0.5	1.05	0.75
Watermelon	0.5	1.00	0.75
Roots and Tubers			
Potato	0.5	1.15	0.75
Sugar Beet	0.35	1.20	0.70
Legumes (Leguminosae)			
Beans, green	0.5	1.05	0.9
Chick pea	0.4	1.00	0.35
Oil Crops			
Sesame	0.35	1.10	0.25
Cereals			
Wheat	0.7	1.15	0.25-0.4
Forages			
Clover hay, Berseem	0.4	0.9	0.85

Source: modified from Allen et al. (1998)

6.3 Irrigation interval

The water distribution system is composed of 25 irrigation hydrants in its maximal size (see Chapter 4 and Figure 4-3). Each hydrant simultaneously irrigates 10 farms of 2 ha (1 ha equals 2.4 $feddan$). Because the water-holding capacity of the soil (W_A) is limited, all hydrants have to be irrigated in a predefined **irrigation interval** F ($days$)[6]. An **irrigation shift** is understood in this work as a group of hydrants being irrigated simultaneously. An irrigation shift takes T_{sh} $hours$ to simultaneously irrigate a group of hydrants. An irrigation interval will contain N_{sh} irrigation shifts.

6.3.1 Irrigation interval for the peak water demand period

The length of the irrigation interval F^{peak} (d) will depend on how long water is available for plants in the soil. The longest possible interval will equal the time needed for consuming a given allowed percentage of the soil's total available water in the root zone of plants (TAW). The TAW in the root zone is defined as TAW $= W_A \cdot Z$, where Z is the effective root depth of the plant (Keller and Bliesner, 1990; Allen et al., 1998; Goyal, 2012). After this percentage of TAW in the soil is exhausted, irrigation needs to be triggered. The fraction of TAW that can be depleted from the root zone before irrigating is called the readily available water (RAW) and is given in by RAW $= p \cdot TAW$ (mm), where p is a percentage dependent on the physical characteristics of the each crop (Doorenbos and Kassam, 1986; Keller and Bliesner, 1990; Allen et al., 1998; Goyal, 2012).

The soil water depletion percentage p for no water stress, was assessed for common relevant crops for the Kalabsha region. The p values for many crops can be found together with their maximum root depths in the FAO Irrigation and Drainage Paper No. 56 (Allen et al., 1998). A small relevant excerpt from this publication is given in Table 6.3.

[6] Also called frequency

Table 6.3: Allowed depletion fraction p for several typical crop types

Crop	Maximum Root Depth Z(m)	Depletion Fraction p
Cantaloupe	0.9-1.5	0.45
Cucumber	0.7-1.2	0.50
Pumpkin, Winter Squash	1.0-1.5	0.35
Zucchini	0.6-1.0	0.50
Sweet Melons	0.8-1.5	0.40
Water Melon	0.8-1.5	0.40

Source: Modified Allen et al. (1998)

As mentioned above, the total available water in the crop root zone TAW, equals the soil's available water holding capacity W_A $\left(\mathrm{mm}\cdot\mathrm{m}^{-1}\right)$ multiplied by the crop's effective root depth Z (m),

$$TAW = W_A \cdot Z \tag{6.2}$$

The W_A is the water reservoir of the soil from which plants are able to extract water and is defined as the difference between the soil's field capacity (FC) and the plant's wilting point (WP) both defined in $\mathrm{m}^3\cdot\mathrm{m}^{-3}$. The soil's FC expresses the water that the soil is able hold against gravitational forces after surplus irrigation water is drained. The water content at wilting point WP is the water content of the soil at which the plants can no longer extract water (Doorenbos and Kassam, 1986; Keller and Bliesner, 1990; Allen et al., 1998; Goyal, 2012). The calculation of W_A is done through the following Equation,

$$W_A = 1000\cdot\,(FC - WP)\ \ \mathrm{mm}\cdot\mathrm{m}^{-1}. \tag{6.3}$$

where the factor 1000 transforms the units $\left[\mathrm{m}^3\ \mathrm{m}^{-3}\right]$ to $\left[\mathrm{mm}\cdot\mathrm{m}^{-1}\right]$, see Keller and Bliesner (1990).

Typical soil water characteristics for several soil types can be found in the FAO Irrigation and Drainage Paper No. 56 (Allen et al., 1998), and are repeated here in the modified following Table 6.4.

Table 6.4: Field capacity (FC) and wilting point (WP) for different soil types

Soil type	FC m^3/m^3	WP m^3/m^3
Sand	0.07-0.17	0.02-0.07
Loamy sand	0.11-0.19	0.03-0.01
Sandy loam	0.18-0.28	0.06-0.16
Loam	0.20-0.30	0.07-0.17
Silt loam	0.22-0.36	0.09-0.21
Silt	0.28-0.36	0.12-0.22
Silt clay loam	0.30-0.37	0.17-0.24
Silt clay	0.30-0.42	0.17-0.29
Clay	0.32-0.40	0.20-0.24

Source: mod. Allen et al. (1998)

The Kalabsha soil was analyzed to analyse its chemical and structural characteristics in the framework of the OWARA research project and classified as **sandy loam**. The analysis results can be seen in Appendix F.

In accordance to the characterization given in Table 6.4, average values were calculated for the FC and WP of this type of soil as: $FC = 0.23 \left(\mathrm{m}^3 \cdot \mathrm{m}^{-3}\right)$ and $WP = 0.11 \left(\mathrm{m}^3 \cdot \mathrm{m}^{-3}\right)$. The soil water capacity W_A of the soil is calculated after (6.3) as,

$$W_A = 1000 \cdot (0.23 - 0.11) \tag{6.4}$$

$$W_A = 120 \text{ mm} \cdot \mathrm{m}^{-1} \tag{6.5}$$

Gross application depth of water per irrigation shift

In this work, the maximum net depth of water to be applied per irrigation shift (d_x) is defined after Keller and Bliesner (1990) as the management maximal allowed depletion

(MAD)[7] of the total water available in the crop's root zone (TAW) and given by,

$$d_x = MAD \cdot TAW = MAD \cdot (W_A \cdot Z) \tag{6.6}$$

where,

d_x = maximum net depth of water to be applied per irrigation (mm)

MAD = management maximal allowed water deficit (%) for the crop in question

W_A = the soil's water holding capacity ($\text{mm} \cdot \text{m}^{-1}$)

Z = effective root depth of the relevant crop (m)

For this dissertation a value of 50% is assumed for MAD. According to Keller and Bliesner (1990), having crops with a relatively high market value (which is the case for the vegetables in Kalabsha), it is generally more secure and profitable to trigger irrigation before 50% of the soil water capacity is depleted in the root zone. Typical root depths for relevant crops can be seen in Table 6.3. For Kalabsha an average crop root depth Z of 0.9 m was assumed based on key-expert's opinions. The maximum allowed depletion of TAW can be calculated taken (6.6) and (6.5) as,

$$d_x = 0.5 \cdot (120 \cdot 0.9) = 54 \text{ mm} . \tag{6.7}$$

[7] The Management Maximum Allowed Depletion (MAD) is a similar concept to the maximal allowed depletion percentage (p) of the total available soil water (RAW). The MAD concept also relies on the physical characteristics of the crop and p but adds management (risk aversion) and economic factors for the irrigation strategy. It can be said, that $MAD < RAW$ where risk aversion plays a major role, and $MAD > RAW$ where water stress is a management option (Keller and Bliesner, 1990; Allen et al., 1998). In this work the MAD principle is adopted.

The gross application depth (d)

In this work the depth of water application per irrigation (d) is calculated by dividing the maximum net depth of water to be applied per irrigation by the irrigation efficiency E_f. An average efficiency of 85% was assumed for the drip and sprinkler systems (Keller and Bliesner, 1990).

$$d = \frac{d_x}{E_f} = \frac{54}{0.85} = 63.53 \text{ mm} \tag{6.8}$$

The peak irrigation interval $\left(F^{peak}\right)$

Knowing d, allows the irrigation interval F^{peak} to be easily derived by (6.9), where d is divided through the peak daily crop evapotranspiration ET_c^{peak} (mm). When using d for triggering irrigation, the calculated irrigation interval F^{peak} will give the maximal possible number of days without irrigation in the peak period, for which no crop water stress will occur[8] (Doorenbos and Kassam, 1986; Keller and Bliesner, 1990; Allen et al., 1998; Goyal, 2012). Based on this,

$$F^{peak} = \frac{d}{ET_c^{peak}} = \frac{d}{K_c^{peak} \cdot ET_0^{peak}}. \tag{6.9}$$

The peak reference evapotranspiration ET_0^{peak} was taken from Table 6.1 for the hottest month of June, 14.48 mm $\cdot$ d^{-1}. The K_c^{peak} coefficient was set equal to 1.2 in accordance to the most demanding crop in Table 6.2 (sugar beet).

The crop evapotranspiration for the peak period is calculated as $ET_c^{peak} = K_c^{peak} \cdot ET_0 = 1.2 \cdot 14.48$; i.e. $ET_c^{peak} = 17.38$ mm $\cdot$ d^{-1}. The maximum allowed irrigation interval is calculated accordingly as,

[8] Irrigation can be triggered before the maximum allowed water depletion is reached, in this case the net depth of water to be irrigated d_n will be lower than d_x.

$$F^{peak} = \frac{63.53}{17.38} = 3.65 \text{ d} \tag{6.10}$$

For security reasons it was decided to take the lowest feasible integer and the maximum irrigation interval is set to be 3 *days*.

Average irrigation intervals for the winter and summer season $\left(F^W,\ F^S\right)$

The winter season begins in Kalabsha in September-October and ends around March-April. The summer season starts in March-April until July-August. For calculating the average irrigation intervals in the two seasons, an average crop is assumed based on a representative farm budget (see Chapter 7). The average crop is assumed to have 140 growth cycle days. A growth cycle as in Figure 6-1. The average reference evapotranspiration for the winter season ET_0^W is calculated from Table 6.1 averaging the ET_0 values between October and March: $ET_0^W = 8.04 \text{ mm} \cdot \text{d}^{-1}$. The average reference evapotranspiration for the summer season is: $ET_0^S = 13.2 \text{ mm} \cdot \text{d}^{-1}$. The representative average crop coefficient for the growth cycle is calculated based on Table 6-1, using a weight average based on the percentage of days in the respective growth period multiplied by the period's crop coefficient. An average $\theta_c = 0.78$ is calculated.

The representative average crop evapotranspiration for the winter season is calculated as $ET_p^W = 0.78 \cdot 8.04 = 6.27 \text{ mm} \cdot \text{d}^{-1}$. For the summer season $ET_p^S = 0.78 \cdot 13.2 = 10.30$ $\text{mm} \cdot \text{d}^{-1}$.

The average irrigation interval for the winter season is now estimated as $F^W = \dfrac{d}{ET_c^W} = \dfrac{63.53}{6.27} = 10.13$, which is rounded to 10 days. The average irrigation interval for the summer season is $F^S = \dfrac{d}{ET_c^S} = \dfrac{63.53}{10.3} = 6.16$, rounded to 6 days.

6.4 Irrigation shifts

The estimation of the appropriate number of irrigation shifts N_{sh} is based on the fact that the irrigation time necessary for a shift T_{sh} (h) multiplied by the number of shifts (N_{sh}), must be lower or equal to the total amount of hours available. This means the time available per day for irrigation D_h (h) multiplied by the allowed irrigation interval F (h) (Keller and Bliesner, 1990),

$$\begin{aligned} T_{sh} \cdot N_{sh} &\leq D_h \cdot F^{peak} \\ N_{sh} &\leq \frac{D_h \cdot F^{peak}}{T_{sh}}. \end{aligned} \tag{6.11}$$

Because the drip discharge at the farms is given $\left(q_E = 1.6 \text{ L} \cdot \text{h}^{-1}\right)$, the irrigation time of one shift T_{sh} is fixed in this work and can be directly determined. For this purpose the discharge capacity at each node needs to be set in relation to the whole water demand of the farms at that node. The drip and sprinkler irrigation system design for each farm was computed externally[9] and can be seen in Appendix G. In this design the drip irrigation area is given as a 200×60 m^{2}rectangle composed of 60 drip lines. The space between rows is one meter $(S_r = 1 \text{ m})$, the emitters in each line are space by 0.6 m (S_e). The number of emitters per farm can be estimated as the number of emitters per line multiplied by the number of lines, *i.e.* $\frac{200}{0.6} \cdot 60 = 20,000$ emitters. Given that each emitter has a discharge of $q_E = 1.6 \text{ L} \cdot \text{h}^{-1}$, the total discharge of the drip Section will be 32 $\text{m}^3 \cdot \text{h}^{-1}$. Multiplied by 10 farms, we reach the hydrants drip discharge of 320 $\text{m}^3 \cdot \text{h}^{-1}$. The sprinkler Section of the farm was equally designed to have a discharge of 32 $\text{m}^3 \cdot \text{h}^{-1}$. The whole discharge of a hydrant (q_H) in the water distribution network is accordingly 640 $\text{m}^3 \cdot \text{h}^{-1}$. The total needed pressure head for hydrants operation equals 40 m (see Appendix G).

[9] The main subject of the dissertation is the dimensioning of the water distribution network. The drip and sprinkler irrigation systems are dimensioned in advance, choosing appropriate dripper and sprinkler discharges.

A maximal water demand for each hydrant is estimated by multiplying d by the irrigation area served. Given the hydrants fixed discharge q_H, the irrigation event must be concluded in time T_{sh}.

$$q_H \cdot T_{sh} = (d \cdot 10) \cdot Area \tag{6.12}$$

The factor 10 is multiplies d for converting units from mm into $\mathrm{m}^3 \cdot \mathrm{ha}^{-1}$. Because in this dissertation the drip and sprinkler emitters discharges are fixed in advance, i.e. the hydrant discharge is fixed at 640 $\mathrm{m}^3 \cdot \mathrm{h}^{-1}$. The irrigation time of each shift is also predetermined and given by,

$$T_{sh} = \frac{(d \cdot 10) \cdot Area}{q_H}. \tag{6.13}$$

Substituting values,

$$T_{sh} = \frac{63.53 \cdot 10 \cdot 20}{640} = 19.85 \text{ h} \tag{6.14}$$

The maximal number of possible shifts in the peak period can be easily calculated by (6.11),

$$N_{sh} = \frac{D_h \cdot F^{peak}}{T_{sh}} = \frac{20 \cdot 3}{19.85} = 3.02 \tag{6.15}$$

In this dissertation it was decided to take the lowest feasible integer, i.e. the maximal feasible number of shifts is set to equal three $(N_{sh} = 3)$. Having calculated the number of irrigation shifts, the optimal spatial distribution of simultaneously operating nodes will be established endogenously by the optimization model. The optimal spatial distribution will be the one that maximizes the NPV of the investment over its life-time and an expression of the trade-offs between initial investment costs (pipeline diameters) and diesel costs (friction losses).

The optimal spatial distribution of simultaneously irrigating nodes

The total possible number of different combinations of simultaneously irrigating nodes (N_{dist}) will depend on the maximum number of irrigation shifts and the total number of irrigation nodes on the network (N_n). It means, $N_{dist} = (N_{sh})^{N_n}$, (see Arviza et al., 2003). In the proposed WDN, we have $N_{sh} = 3$, and 25 irrigation nodes, i.e. there are $8.5 \cdot 10^{11}$ possible irrigating node combinations. The problem in this dissertation is even more complex because the size of the network is endogenous, i.e. the total number of irrigation nodes is a variable. Furthermore, the distribution of the simultaneous irrigating nodes is influenced by the initial investment (pipeline diameters) and energy costs trade-offs.

6.5 Average total hours of irrigation per year

As discussed above, the system will be designed to operate on three shifts ($N_{sh} = 3$) as calculated for the maximal allowed irrigation interval in the peak demand period of the year ($F^{peak} = 3$ d). Out of the peak demand period, the system can be operated according to different possible irrigation intervals depending on the management strategy. In this work, and for calculating yearly total hours of operation of the system, average irrigation intervals for the two crop seasons were calculated in Section 6.3. A new irrigation cycle will on average only start in the 10^{th} day for the winter season, and start on the 6^{th} day in the summer season. For example in the winter season the system is 7 days off, and in the summer season only 3 days off, on average. Given these assumptions, the total hours of operation for the system can be calculated as:

$$Thrs = T_{sh} \cdot N_{sh} \cdot \left(\frac{1}{6} + \frac{1}{10}\right) \cdot 140 = 19.85 \cdot 3 \cdot \left(\frac{1}{6} + \frac{1}{10}\right) \cdot 140 = 2223.2 \text{ h} \tag{6.16}$$

6.6 Summary

In this Chapter the Penman-Monteith method for calculating ET_0 and ET_p was introduced. The ET_0 was calculated using CROPWAT 8.0 for all months of the year and for the Kalabsha climatic conditions. The parameter ET_p^{peak} was calculated for the most demanding crop and hottest month of the year. ET_p^{peak} is the relevant parameter for appropriate dimensioning of the system, it will assure compliance of water delivery in every other month of the year where water demands are lower. Based on the Kalabsha calculated soil-water holding capacity W_A and management factors, the maximal gross depth of water to be delivered to the fields d could be calculated. Based on d and ET_p^{peak}, the maximum irrigation interval for the peak season $\left(F^{peak}\right)$ was estimated. Based on the maximal allowed irrigation interval, it was possible to estimate the maximal allowed number of irrigation shifts N_{sh} imposed in the model for dimensioning the network. For calculating the operating costs (diesel), average irrigation intervals for the winter $\left(F^W\right)$ and summer season $\left(F^S\right)$ were also estimated. Based on these estimates, the total number of operating hours could be calculated. Table 6.5 presents an overview of the calculated parameters.

Table 6.5: Summary of estimated parameters

Name	Symbol	Estimate	Units
Peak crop evapotranspiration	ET_0^{peak}	17.4	mm/d
Field capacity	FC	0.23	m^3/m^3
Wilting point	WP	0.11	m^3/m^3
Soil water capacity	W_A	120	mm/m
Gross applic. depth	d	63.53	mm
Irrigation interval (peak)	F^{peak}	3	d
Winter irrigation interval	F^W	10	d
Summer irrigation interval	F^S	6	d
Max. number of shifts	N_s	3	shifts
Total hours of irrigation per year	$Thrs$	2223	h
Time of an irrigation shift	T_sh	19.85	h

Chapter 7

The economic framework

7.1 Introduction

The proposed optimization model is applied to a case-study in Egypt. As in many other developing countries, markets and prices in Egypt are often characterized by strong economic distortions or direct governmental market interventions. Biased prices need to be corrected through *shadow prices that* express, as well as possible, the true costs and benefits accruing to the implementing agency (Brent, 2000)[1]. It is believed that subventioning of the diesel price is by far the most relevant market distortion for the analysis of this dissertation's case-study. Diesel prices in Egypt are below world prices for crude oil (GIZ, 2012). The use of domestic market prices for diesel would represent a serious bias to the model's CBA. With regard to the investment costs, pumps and farm drip and sprinkler irrigation systems, it is assumed in this dissertation that the Egyptian market prices for these tradables are close enough to world market prices and the eventual bias is small enough when compared with the main problem of the valuation, i.e. the diesel prices.

The remainder of the chapter introduces, in Section 7.2, the general form of the model's objective function and identifies the benefits and costs of the investment. In this section the

[1] Classical approaches in project appraisal are the works of UNIDO (1972), Little and Mirrlees (1974) and Squire and Tak (1975).

discounting procedure for benefits and costs of the investment is also discussed considering real escalating rates of diesel prices and the project's benefits during the investment's life time. In Section 7.3 the highly subventioned fuel prices in Egypt are discussed, a *shadow* price for diesel is derived together with a reference value for future diesel price escalation rates. In Section 7.4 expected net-benefits generated by the agricultural activity at each hydrant are presented, as well as a discussion about the growth rates of the hydrants' future benefits. Section 7.5 presents the market prices for PVC pipes in Egypt. In Section 7.6 the pumping plant cost component is estimated through an empirical relationship between power delivered and the price of typical centrifugal pumps in the Egyptian market. The relationship is used to charge the objective function with a price for the pumping plant that depends on the power demanded from the system. Section 7.7 introduces the costs involved in implementing drip and sprinkler irrigation systems for each of the ten farms connected to each irrigation hydrant. The chapter closes with Section 7.8 introducing a final formulation of the model's objective function.

7.2 The model's objective function and discounting procedures

The model's objective function expresses the net present value (NPV) of the discounted irrigation hydrants' benefits[2], initial investments, energy costs (EC), O&M costs[3], depreciation and the salvage value of the network at the end of the investment period. The model will include more irrigation hydrants augmenting the size of the network, as long as their marginal contribution to the objective function is at least as great as their marginal costs. The constraints to the maximization problem basically comprise irrigation management and hydraulic constraints.

[2] Farm's net-benefits.

[3] Non-energy O&M costs.

For the proposed optimization model the following NPV *general* objective function is proposed:

$$
\begin{aligned}
NPV &= \sum_{ij\in H}\sum_{t=1}\frac{B(i,j)_t}{(1+r)^t} - \sum_t \frac{EC_t}{(1+r)^t} - \sum_{ij\in A} PC(i,j) \\
&\quad - \sum_t \frac{\text{O\&M}}{(1+r)^t} - \widehat{\beta}\cdot PW - \sum_{ij\in H} IS(i,j) \\
&\quad - \sum_{t=1}\frac{D}{(1+r)^t} + \sum_{t=1}\frac{SV_T}{(1+r)^t}
\end{aligned} \tag{7.1}
$$

where,

i = beginning of a pipe section

j = end of a pipe section

t = time index $\{1, 2, ...T\}$

A = set of all pipe sections

H = set of pipe sections with an irrigation hydrant at j

r = interest rate

NPV = net present value of the water distribution network investment [EGP]

$B(i,j)_t$ = the benefits generated by each irrigation hydrant j in year t [EGP]

EC_t = energy costs (diesel costs) for irrigation of all hydrants in year t [EGP]

$O\&M$ = operation and maintenance costs[4] of the system per year [EGP]

$PC(i,j)$ = initial pipeline investments costs [EGP]

$\widehat{\beta}$ = estimated empirical coefficient of pump cost per unit power [EGP/kW]

PW = the network's demanded power [kW]

$IS(i,j)$ = the initial investment costs in hydrants for farms irrigation systems [EGP]

D_t = the system's straight-line yearly depreciation [EGP]

[4]Non-energy

SV_T = salvage value of the system, in terminal year T $[EGP]$

The *first* term of Equation (7.1) is the discounted sum of aggregated benefits of the hydrants accruing from the agriculture activity of the 10 farms connected to each hydrant.

The *second* and *third* terms are the discounted energy costs and the initial investment costs in the network's pipelines, The energy costs (EC) reflect the diesel costs of pumping water in the two cropping seasons of the year. The *fourth* element is the discounted non-energy O&M costs. The yearly non-energy O&M cost estimation is based on common practice, i.e. a yearly cost was set of 0.5% for the initial pipeline network investment and 2% for the pumping system (Trifunovic, 2006). The non-energy O&M discounted costs were modelled as:

$$\sum_t \frac{\text{O\&M}}{(1+r)^t} = \sum_t \frac{0.005 \cdot \sum_{ij\,\in\,H} PC\,(i,j)}{(1+r)^t} + \sum_t \frac{0.02 \cdot \left(\widehat{\beta} \cdot PW\right)}{(1+r)^t}$$

The *fifth* cost component represents the system's pump cost. This cost was estimated based on the empirical relationship between demanded power and market pump costs (see Section 7.6).

The *sixth* cost element are the investments at the hydrant level (10 farms) for equipping each farm with 1.25 ha drip irrigation and 0.75 ha sprinkler irrigation systems (see Appendix G).

The *seventh* and the *eight* terms are the discounted depreciation costs and salvage value of the network. For D_t a simple yearly constant depreciation D is assumed over the life-time of the network, i.e. the difference between the initial investment costs and the salvage value of the network divided by the operation's life-time (n). The discounted constant depreciation is calculated as:

$$\sum_t \frac{D}{(1+r)^t} = \frac{\left(\sum_{ij\,\in\,H} PC\,(i,j) + \widehat{\beta} \cdot PW - SV_T\right)}{n} \cdot \sum_t \frac{1}{(1+r)^t}$$

The salvage value of the network is assumed to be too low to be considered in the model, i.e. $SV_T = 0$. The materials will be obsolete after the project's life-time. The local market conditions of this remote area would not allow any considerable value recovery of the remaining network materials (personal experience in the region). The salvage value component is dropped hereafter from the objective function.

7.2.1 Discounting constant cash-flows

If one could assume constant benefits and energy costs during the project's life-cycle, the present value of such uniform *cash-flows* (CF) would be equivalent to a simple annuity. The respective present value (PV) is given by,

$$PV = \sum_{t=1} \frac{CF}{(1+r)^t} = CF \cdot \sum_{t=1} \frac{1}{(1+r)^t} \ . \tag{7.2}$$

The term $\sum_{t=1} \frac{1}{(1+r)^t}$ is equivalent to the sum of a geometric series of reason $\lambda = \frac{1}{1+r}$, where $\lambda \neq 1$ (McCutcheon et al., 1989; Broverman, 2010; Rogers and Duffy, 2012). It can be shown, that the sum of the n first terms of this geometric series is,

$$S_n = \lambda + \lambda^2 + \lambda^3 + ... + \lambda^n = \sum_{k=1}^{n} \lambda^k \ . \tag{7.3}$$

When multiplying (7.3) by $(1 - \lambda)$ gives,

$$S_n = \frac{\lambda \cdot \left(1 + \lambda + \lambda^2 + \lambda^3 + ... + \lambda^{n-1}\right) \cdot (1 - \lambda)}{(1 - \lambda)} \ . \tag{7.4}$$

The summation term in the numerator will cancel to $(1 - \lambda^n)$ after the multiplying with $(1 - \lambda)$, and we can simplify (7.4) as,

$$S_n = \frac{\lambda (1 - \lambda^n)}{1 - \lambda}. \tag{7.5}$$

This means the sum of the n first terms of a stream of constant cash-flows converges to the expression $\dfrac{\lambda\,(1-\lambda^{n})}{1-\lambda}$ (McCutcheon et al., 1989; Broverman, 2010). The discounted cash-flow can be expressed as,

$$PV = CF \cdot \frac{\lambda\,(1-\lambda^{n})}{1-\lambda} \; . \tag{7.6}$$

Given the fact that $\lambda = \dfrac{1}{1+r}$,

$$PV = CF \cdot \frac{1}{1+r} \cdot \frac{1-\left(\dfrac{1}{1+r}\right)^{n}}{1-\dfrac{1}{1+r}} \; , \tag{7.7}$$

with further manipulation it can be shown that,

$$PV = CF \cdot \frac{(1+r)^{n}-1}{r \cdot (1+r)^{n}} \; , \tag{7.8}$$

(for similar derivations see: Dihllon, 1989; McCutcheon et al., 1989; Keller and Bliesner, 1990; Kellison, 2008; Prabhata et al., 2008; Broverman, 2010; Rogers and Duffy, 2012).

7.2.2 Discounting growing cash-flows

International energy and food prices' historical time-series show evidence of increasing growth rates of real prices. The price escalation rates (e) in the future need to be accounted for when setting the objective function of the model for balancing initial investments, future recurrent benefits and energy costs (Keller and Bliesner, 1990). The present analysis refers to real interest (r) and real escalation rates (e)[5].

Let us consider the case of discounting a cash-flow (CF) with a real escalation rate e:

[5] The prices used for the initial investments, energy costs (diesel) and food prices (irrigation hydrant's net-benefits) are all refered to the constant base year 2010. All calculations hereafter are done for real values, real changes in prices.

$$PV = \sum_{t}^{n} \frac{CF\,(1+e)^{t-1}}{(1+r)^{t}} \ .$$

In this expression the real prices are allowed to change at an annual escalating rate e (as opposed to the uniform CF formulation). In fact, when considering an escalating rate, we are dealing with a present value of a growing annuity. This can be shown as,

$$PV = \frac{CF}{1+r} + \frac{CF\,(1+e)}{(1+r)^2} + \frac{CF\,(1+e)^2}{(1+r)^3} + ... + \frac{CF\,(1+e)^{n-1}}{(1+r)^n} \ , \qquad (7.9)$$

which represents discounted future cash-flows, that grow at an escalating rate e. The PV of this growing annuity can be expressed by,

$$PV = \frac{CF}{1+r} + \frac{CF}{1+r}\left(\frac{1+e}{1+r}\right) + \frac{CF}{1+r}\left(\frac{1+e}{1+r}\right)^2 + ... + \frac{CF}{1+r}\left(\frac{1+e}{1+r}\right)^{n-1} \ . \qquad (7.10)$$

Rearranging terms, and considering that we now have a geometric series of ratio $\lambda = \frac{1+e}{1+r}$, where $\lambda \neq 1$ and $e < r$, we have,

$$PV = \frac{CF}{1+r} \cdot \left(1 + \left(\frac{1+e}{1+r}\right) + \left(\frac{1+e}{1+r}\right)^2 + ... + \left(\frac{1+e}{1+r}\right)^{n-1}\right) \ , \qquad (7.11)$$

which equals,

$$PV = \frac{CF}{1+r} \cdot \left(\frac{1 - \left(\frac{1+e}{1+r}\right)^n}{1 - \frac{1+e}{1+r}}\right) \ . \qquad (7.12)$$

Rearranging terms gives the model's final expression for discounting cash-flows at constant 2010 prices with a real escalation rate e :

$$PV = CF \cdot \frac{(1+r)^n - (1+e)^n}{(1+r) - (1+e)} \cdot \frac{1}{(1+r)^n} \ . \tag{7.13}$$

This is a formula normally used in many energy economics references for inclusion of the escalating costs of energy in investment analysis (this section presented an own derivation of the end formulas normally found in the literature, for example: Dihllon, 1989; McCutcheon et al., 1989; Keller and Bliesner, 1990; Kellison, 2008; Prabhata et al., 2008; Broverman, 2010; Rogers and Duffy, 2012.

7.3 Estimation of annual energy costs

The whole power demanded by the characteristic curve of the system was derived in Chapter 5. Equation 5.26 is repeated below:

$$PW = \frac{\rho g \cdot Q_{pu} P_{pu}}{\eta_{d-w}} \ \text{[kW]} \ , \tag{7.14}$$

where again, ρ is the density of water, 1000 $\text{kg} \cdot \text{m}^{-3}$, g is the acceleration of gravity in $\text{m} \cdot \text{s}^{-2}$, Q_{pu} and P_{pu} are the operating discharge and pressure of the pump system given in $\text{m}^3 \cdot \text{s}^{-1}$ and m respectively. The whole combined diesel-to-water efficiency of the aggregate (diesel generator)-(electrical motor)-(centrifugal pump) is given by η_{d-w} (see Chapter 5)[6].

The calculated total yearly demanded power in kWh is transformed to *diesel consumption* (L) by multiplying PW with the total operating hours of irrigation per year ($Thrs$) and with an appropriate conversion coefficient $cf = 0.23 \ \text{L} \cdot \text{kWh}^{-1}$[7]. The total energy costs (EC) for the cropping seasons, are calculated by multiplying with the diesel price p_d in $EGP \ \cdot \text{L}^{-1}$,

[6] For simplicity η_{d-w} will from now on be referred to as η.

[7] This figure was estimated as an average for the type of generators found in the Egyptian market opearting on 3/4 of maximal load.

$$EC = \left[\frac{\rho g \cdot Q_{pu} P_{pu}}{\eta}\right] \cdot Thrs \cdot cf \cdot p_d \quad [EGP] \tag{7.15}$$

As discussed in Chapter 6 an average representative crop cycle of 140 days for each of the two crop seasons was assumed and the average irrigation intervals for the summer and winter seasons $\left(F^S \text{ and } F^W\right)$ were estimated. The total number of operating hours a year was calculated as: $Thrs = T_{sh} \cdot N_{sh} \cdot \left(\frac{1}{F^S} + \frac{1}{F^W}\right) \cdot 140$.

The energy cost equation above can be reformulated as,

$$EC = \left[\sum_s Q_{pu}(s) \cdot P_{pu}(s)\right] \cdot \frac{\rho g}{\eta} \cdot \left(T_{sh} \cdot \left(\frac{1}{6} + \frac{1}{10}\right) \cdot 140\right) \cdot cf \cdot p_d \tag{7.16}$$

Defining the parameter $\Phi = \left[\frac{\rho g}{\eta} \cdot \left(T_{sh} \cdot \left(\frac{1}{6} + \frac{1}{10}\right) \cdot 140\right) \cdot cf \cdot p_d\right]$, the final expression for discounting EC with a diesel price escalating rate e_d is:

$$PV(EC) = \sum_s \Phi \cdot \left[Q_{pu}(s) \cdot P_{pu}(s)\right] \cdot \left[\frac{(1+r)^n - (1+e_d)^n}{(1+r) - (1+e_d)} \cdot \frac{1}{(1+r)^n}\right] \tag{7.17}$$

7.3.1 Estimation of a shadow price for Egyptian diesel

Egypt is a country where fuel prices are heavily subsidized. According to GIZ (2012), Egypt belongs to the group of countries within the "Fuel Taxation Category 1: Very High Fuel Subsidies". In this category the average retail price for fuel is below the world prices for crude oil. In Table 7.1 Egyptian diesel retail prices in $\text{USD} \cdot \text{L}^{-1}$ are compared with retail diesel prices for the US benchmark[8].

The difference of prices is enormous and clearly highlights the necessity for a shadow

[8] According to GIZ (2012, p. 24), the presented prices are defined as: "Retail price of diesel and gasoline in the United States. Cost-covering retail prices incl. industry margin, VAT and incl. approx. 10 cents for two road funds (federal and state). This fuel price being without any other specific fuel taxes may be considered as the international minimum benchmark for a non-subsidized road transport policy."

Table 7.1: Retail prices for diesel in USD/L

Year	Egypt	U.S.
1991	0.07	na
1993	0.09	0.28
1995	0.12	0.33
1998	0.12	0.27
2000	0.10	0.48
2002	0.08	0.39
2004	0.10	0.57
2006	0.12	0.69
2008	0.20	0.78
2010	0.32	0.84

Source: GIZ (2012)

price for diesel in the present economic analysis. Shadow prices could be built on international diesel or crude oil prices, i.e. 'free on border' (fob) or the 'cost, insurance and freight' (c.i.f), depending on whether Egypt is considered a net exporter or net importer of diesel or crude oil. To these prices other domestic costs should also be added or subtracted covering any domestic services (Brent, 2000). Given that no data was available on either f.o.b or c.i.f prices, it was decided in this dissertation to derive a *proxy* for the shadow price of diesel based on the referred US benchmark. The US diesel prices showed in Table 7.1 above are considered a benchmark for non-subsidized fuels (GIZ, 2012), and taken in this work as the world market price for retail diesel. The US diesel price is converted to Egyptian Pounds (EGP) using the prevalent average exchange rate of 2010 (1 USD $\simeq$ 7 EGP). Given the US 2010 price of 0.84 $\mathrm{USD} \cdot \mathrm{L}^{-1}$, the Egyptian diesel price would be 5.88 $\mathrm{EGP} \cdot \mathrm{L}^{-1}$.

> This diesel price is only a reference value, a *proxy for* the proposed optimization model, where any kind of parameterization of prices can be performed. Nevertheless, this diesel price gives a 'feeling' for the true social costs that the Egyptian Government would incur in powering such a water distribution network.

7.3.2 Estimation of escalating rates of diesel shadow prices

If we are to discount diesel prices from 20, or 30 years from now, we need to understand how these prices could develop in this time period. Diesel prices frequently change over time, are highly volatile, and the escalation rate is normally different from the rate of inflation, e.g. GDP Deflator or Consumer Price Index (CPI). According to Keller and Bliesner (1990), the escalation of energy prices needs to be accounted for when designing irrigation water distribution networks. In order to estimate the potential magnitude of world diesel price escalation rates in the future projected diesel price indices for the U.S. transportation sector from 2011-2040 were used in this dissertation. The projection serves as a guide, upon which basis parameterizations in the model can be performed, allowing the construction of realistic scenarios for changes in world diesel prices (escalation rates).

The following projected price indices were estimated by the Energy Information Administration (EIA) of the U.S. Department of Energy. The energy prices through to 2035 were generated by EIA using the US National Energy Modeling System (NEMS), (USDE, 2011) [9].

The projected US fuel price indices were estimated relative to the base year 2010. For example, the projections are constant Dollar estimates (base year 2010) and can be seen in the following Table 7.2.

The escalation rates of fuel prices for the U.S. can now be calculated from the data in Table 7.2 using the following compounding formula:

$$F = P(1 + e_d)^n \tag{7.18}$$

[9] NEMS is an energy market model and supports the U.S. Energy Department in market evaluation and decision making in evaluating the impacts of alternative energy policies or assumptions on energy markets: "NEMS produces projections of the U.S. energy future, given current laws and policies and other key assumptions, including macroeconomic indicators from Data Resources, Inc., the production policy of the Organization of Petroleum Exporting Countries, the size of the economically recoverable resource base for fossil fuels, and the rate of development and penetration of new technologies.", (USDE, 2011).

Table 7.2: Projected U.S. fuel price indices (excluding general inflation), for the transportation sector

2011	1.03	2026	1.44
2012	1.08	2027	1.46
2013	1.16	2028	1.48
2014	1.22	2029	1.49
2015	1.25	2030	1.50
2016	1.28	2031	1.52
2017	1.30	2032	1.54
2018	1.33	2033	1.55
2019	1.34	2034	1.57
2020	1.36	2035	1.59
2021	1.37	2036	1.61
2022	1.39	2037	1.63
2023	1.40	2038	1.65
2024	1.41	2039	1.66
2025	1.43	2040	1.68

Source: United States Department of Energy (2011)

where F represents the future value of the energy price index, P is the actual or present value of the energy price index and e_d represents the escalation rate over n years.

Solving for e_d and multiplying per 100 to obtain percentage changes, we get

$$e_d = \left(\sqrt[n]{\frac{F}{P}} - 1 \right) \cdot 100. \tag{7.19}$$

The calculated escalation rates e_d can be seen in Table 7.3 for different single periods (e.g. 5 year periods) and for the whole estimation time window 2011-2040 (30 years).

Table 7.3: Escalation rates for U.S. diesel in the transportation sector

Period	Escal. Rate
2011-2015	5.0
2015-2020	1.7
2020-2025	1.0
2025-2030	1.0
2030-2035	1.2
2035-2040	1.1
2011-2040	1.6

7.4 Estimation of benefits per irrigation hydrant

According to the Ministry of Agriculture and Land Reclamation (MALR), the Ministry of Planning (MoP) and local key -experts, the suitable crop types for the climate in this region are: cereals, vegetables, oilseeds and spices (UNDP, 2002). Given that the soils in the region are sandy soils with very low organic matter (see soil analysis in Appendix F), the farming system should include animal husbandry and use manure for enrichment of soil structure and organic content. Family needs for protein is another concern of planners in this region (UNDP, 2002). Against this background animal raising is considered an important activity and fodder cultivation e.g. clover, alfalfa, needs to be considered in the cropping pattern. The calculation of the irrigation net benefits is based on a representative farm budget of 2 ha (ca. 5 feddan). The farm budget can be seen in Table 7.4 and is based on key expert interviews, own survey results, as well as on secondary studies from the Egyptian Ministry of Planning, Governorate of Aswan (UNDP, 2002). The budget shows a net-benefit estimation of $39,371$ EGP per year for each farm. Given that each hydrant connects ten farms, the whole agricultural benefit of a network irrigation hydrant equals

393, 710 EGP per year[10] (winter and summer seasons).

Table 7.4: Net-benefit of a representative Kalabsha farm

	Area (feddan)	Net Return (EGP)	Labor requirements (man-days)
Winter			
Tomato	1	10,846	104
Cucumber	1	4,973	71
Broad bean	1	2,183	34
Wheat	1	2,238	34
Total winter	4	20,240	243
Summer			
Coriander	0.5	2,565	32
Cumin	0.5	2,808	76
Hibiscus/med. plants	1	8,059	160
Total summer	2	13,431	268
Perennial: Clover	1	5,700	36
Grand total	7	39,371	388

Source: Modified UNDP (2002)

[10] Ca. 49,214 EUR/year for 10 farms given an exchange rate of 8 EGP/EUR

7.4.1 Escalation rates of farms' incomes (hydrants net-benefits)

The yields considered for the farm budget are expected potential yields for this region's climate and soils. The whole farm income is given for 2010 base year prices. The farming production in Kalabsha is market oriented, having Aswan city as the main target. It is assumed in this work that real incomes of the settlements families will gradually increase during the time window of the investment. The estimation of an empirical, significant, real growth rate is not in the scope of this dissertation. Nevertheless, real growth rates for farms' incomes will be assumed based on plausibility considerations related to the development of international food markets. For this purpose, it is assumed that the prices for agricultural products in Egypt are strongly correlated with international food prices (especially wheat). The escalation rates of these international food prices are taken in this work only as a reference for potential farm income growth scenarios in the optimization runs (growth of hydrants' benefits during the project's life-time).

In Table 7.5 we can see the development of the aggregated FAO Food Price Index until 2013. According to FAO (2013)[11], the index consists of:

> '... the average of 5 commodity group price indices mentioned above weighted with the average export shares of each of the groups for 2002-2004: in total 55 commodity quotations considered by FAO commodity specialists as representing the international prices of the food commodities noted are included in the overall index'.

Applying Equation 7.19 the FAO international food price index between 1990 and 2013 returns a real escalating rate for food prices of 1.03. This value is taken together with the escalating rate for diesel prices from Section 7.3.1 as reference values only, upon which the model's optimization scenarios are built.

[11] All FAO indices have been deflated using the World Bank Manufactures Unit Value Index (MUV) rebased from 1990=100 to 2002-2004=100. For more information on the indices construction see FAO at: http://www.fao.org/worldfoodsituation/wfs-home/foodpricesindex/en/

Table 7.5: Annual Real Food Price Indices (2002-2004=100)

	Date	Food	Meat	Dairy	Cereals	Oils	Sugar
1	1990	103.2	121.3	73.2	95.6	72.4	174.3
2	1991	102.4	123.9	78.6	95.9	78.2	125.7
3	1992	105.4	121.6	92.7	99.5	81.9	124.8
4	1993	101.4	114.5	82.0	96.5	83.3	137.9
5	1994	107.5	111.8	80.0	101.6	110.2	167.0
6	1995	109.6	105.4	97.6	106.2	111.3	167.8
7	1996	118.5	117.9	100.4	129.3	102.1	155.8
8	1997	116.3	120.9	103.2	110.3	110.5	158.4
9	1998	111.8	107.8	103.5	104.3	135.6	132.2
10	1999	94.8	100.4	88.6	93.0	94.0	91.3
11	2000	92.9	98.5	98.1	87.6	69.7	119.3
12	2001	101.4	104.8	116.3	94.0	73.4	133.1
13	2002	97.8	97.5	89.5	102.8	94.7	106.4
14	2003	98.0	97.0	95.4	98.4	101.1	100.8
15	2004	103.7	104.9	113.1	99.1	103.5	93.8
16	2005	103.3	105.8	119.2	91.2	91.3	123.6
17	2006	108.2	101.2	109.3	103.9	96.0	179.0
18	2007	127.7	100.7	170.9	134.3	136.8	115.1
19	2008	147.6	113.2	162.2	175.6	167.8	134.2
20	2009	123.9	105.0	111.8	137.2	119.2	203.2
21	2010	139.4	114.6	150.8	137.4	146.1	227.3
22	2011	154.0	119.5	149.2	167.0	170.7	249.7
23	2012	141.5	117.0	126.1	161.1	150.6	204.3
24	2013	140.2	117.9	138.7	163.0	135.6	174.7

Source: FAO at http://www.fao.org/worldfoodsituation/wfs-home/foodpricesindex/en/

Discounting irrigation benefits accounting for potential income escalating rates

The expression for discounting net-benefits at constant 2010 prices is given by the following expression using the same formulation of the discounting factor, but now using the proxy food price escalation rate e_f for the benefits:

$$PV\left(B\right) = \sum_{ij\ \in\ N} B\left(i,j\right) \cdot \left[\frac{\left(1+r\right)^n - \left(1+e_f\right)^n}{\left(1+r\right) - \left(1+e_f\right)} \cdot \frac{1}{\left(1+r\right)^n}\right] \tag{7.20}$$

7.5 Pipeline costs of the distribution network

The pipe costs for different diameters and pressure classes were assessed for the Egyptian market. The following Table 7.6 is representative for PVC pipes for pressure up to 6 bar, a class suitable for the head ranges of the present water distribution network. The price of PVC in Egypt was 10,000 EGP per ton for the 2010 base-year.

Table 7.6: Pipe prices for standard Egyptian PVC pipes of class 6 bar

Diameter mm	Thickness mm	Mass kg/m	Price EGP/m
40	1.8	0.334	3.34
50	1.8	0.422	4.22
63	1.9	0.562	5.62
75	2.2	0.782	7.82
90	2.7	1.13	11.3
110	3.2	1.64	16.4
125	3.7	2.13	21.3
140	4.1	2.65	26.5
160	4.7	3.44	34.4
180	5.3	4.37	43.7
200	5.9	5.37	53.7
225	6.6	6.76	67.6
250	7.3	8.31	83.1
280	8.2	10.4	104
315	9.2	13.2	132
355	10.4	16.7	167
400	11.7	21.1	211
450	13.2	26.8	268
500	14.6	32.9	329
560	16.4	41.4	414
630	18.4	52.2	522
710	20.7	66.1	661

Source: Prices provided by the Egyptian company GM UPVC pipes

7.6 Deriving pump system costs

The pump system cost is another component of the initial investments. This cost is estimated in this work using the positive empirical relationship between the power delivered by a pump and its price[12]. The relationship is estimated by regressing the costs of a variety of centrifugal pumps with different impeller diameters on the power these pumps can deliver. The data for this purpose was collected from the characteristic curves of each pump. The characteristic curves of each pump are normally delivered by pump manufacturers (see Figure 5-2). The company *Allweiler & Farid* kindly facilitated all the prices and characteristic curves of the presented pumps. From each pump type and characteristic curve, the delivered discharge, pressure, and consumed power were collected together with the pump's price. These parameters can be seen in Table 7.8 for the points of maximal operating efficiency. The regression model is showed in Equation 7.21 with estimates presented in Table 7.7

$$PRICE = \alpha + \beta POWER + \varepsilon \tag{7.21}$$

Table 7.7: Empirical relationship between prices and pump power

	PRICE
POWER	287.6
	(5.78)
Adj. R^2	0.637
n	20

The best model delivered the estimates $\widehat{\alpha} = 0$, and $\widehat{\beta} = 287.6$ [EGP/kW], where the value in brackets is the $t-statistic$, showing a very high significance for the assumed normal

[12] The pump price includes the price of the aggregates

distribution. The term $Adj.R^2$ is the adjusted regression's coefficient of determination showing a very acceptable proportion of explained variance.

Table 7.8: Pump prices for 'Allweiler and Farid' centrifugal pumps

Pump type NT	Rotations rpm	Impeller mm	Efficiency perc.	Q m^3/h	P m	Power kW	Price EGP
150-400	1750	408	75	400	75	105	12,285
150-250	1450	280	82	365	19.8	24	17,270
100-200	2900	215	80	290	49.5	48	20,230
125-315	1450	328	84	210	34	22.5	20,615
150-315	1450	328	82.5	360	32.5	39	22,600
125-400	1450	408	77	280	48	48	26,445
200-315	1750	340	85	610	50	100	27,125
80-250	2900	265	75	205	87.5	65	28,170
250-315	1750	355	84	1000	45	150	30,750
100-250	2900	260	75	290	80	80	31,800
200-400	1750	420	83	790	73	183	32,550
150-400	1450	408	80	360	50	61	34,985
250-400	1750	420	82.5	1160	62.5	243	38,000
250-400	1750	400	82.5	1100	55	240	38,000
150-250	1750	280	82	440	29	42	46,200
250-315	1450	355	84	850	31	87	57,000
150-315	1750	328	82.5	430	47.5	70	58,650
200-400	1450	420	83	650	48	105	77,000
250-400	1450	420	82.5	960	47	135	87,500

Source: Data kindly provided by the company Allweiler and Farid

7.7 Costs of implementing farm level irrigation systems

The total costs of equipping 10 farms with a drip and sprinkler irrigation system connected to each hydrant is approximately $390,800$ EGP. Cost calculations can be seen in Appendix H.

7.8 Model's objective function revisited

Given the above exposition we can now reformulate the general objective function of the investment model of this dissertation.

$$
\begin{aligned}
NPV \;=\; & \sum_{ij\;\in\;H} B\left(i,j\right)\cdot\left[\frac{\left(1+r\right)^{n}-\left(1+e_{f}\right)^{n}}{\left(1+r\right)-\left(1+e_{f}\right)}\cdot\frac{1}{\left(1+r\right)^{n}}\right] \\
& - EC\cdot\left[\frac{\left(1+r\right)^{n}-\left(1+e_{d}\right)^{n}}{\left(1+r\right)-\left(1+e_{d}\right)}\cdot\frac{1}{\left(1+r\right)^{n}}\right] \\
& - \sum_{ij\;\in\;A} PC\left(i,j\right)\cdot\left[1+\left(0.005+\frac{1}{n}\right)\cdot\frac{\left(1+r\right)^{n}-1}{r\left(1+r\right)^{n}}\right] \\
& - \widehat{\beta}\cdot PW\cdot\left[1+\left(0.02+\frac{1}{n}\right)\cdot\frac{\left(1+r\right)^{n}-1}{r\left(1+r\right)^{n}}\right] \\
& - \sum_{ij\;\in\;N} IS\left(i,j\right)
\end{aligned}
$$

where $EC = \sum_{s}\Phi\cdot\left[Q_{pu}\left(s\right)\cdot P_{pu}\left(s\right)\right]$.

The objective function expresses an interesting trade-off between future developments on the diesel and food markets. The results will very much depend on the different assumptions about e_d, e_f and the interest rate r.

7.9 Summary

This chapter showed how the optimization model for dimensioning WDN was cast in a CBA framework. The discounting procedure for recurrent costs and benefits was introduced. The diesel prices in Egypt were analyzed as the most relevant market distortion for the analysis. The US benchmark for diesel prices was taken as the most suitable *shadow* diesel price. Future diesel cost developments are accounted for in the discounting process by considering appropriate escalation price rates. Reference values for escalation rates of world diesel prices were estimated from secondary data for the US benchmark.

Relative to the model's net-benefits, a representative expected farm budget was presented assuming potential yields for the climate and soil conditions of the region. For future real net-benefit growth rates it was assumed that these are closely related with world food price developments. The escalating rate of international food prices is taken as a proxy for the net-benefit developments in the discounting process.

Table 7.9: Summary of estimated parameters

Name	Symbol	Estimate	Units
Irrigation nodes benefits	$B(i,j)$	393,710	EGP
Initial inv. costs in pipelines	$PC(i,j)$	Table 7.6	EGP
Pump cost regression coeff.	β	287.6	EGP/kW
Diesel shadow price	p_d	5.88	$\text{EGP} \cdot \text{L}^{-1}$
Ref. esc. rate for diesel prices	e_d	1.6	%
Ref. esc. rate for food prices	e_f	1.03	%
Initial inv. costs per hydrant	$IS(i,j)$	390,800	EGP

Part III

Optimization Model

Chapter 8

The MINLP formulation of the WDN dimensioning problem

8.1 Introduction

This chapter introduces the original MINLP mathematical formulation of the maximization problem. This is a convex MINLP form, where the objective function is shown to be non-linear and concave and the constraint set is convex, containing non-linear but convex functions (law of energy conservation with the Hazen-Williams equation). MINLP problems are NP-hard because they are generalizations of MILP problems, which are classified as being NP-hard (for detailed discussions on NP-Completeness and time complexity see: Garey and Johnson, 1979; Schrijver, 1986; Wolsey, 1998; Nemhauser and Wolsey, 1999).

The remainder of this chapter introduces in Section 8.2 the objective function of the model. In Section 8.3.1 the law of energy conservation dealt with in the hydraulic foundations of Chapter 5 is reformulated and its non-linear and convex constraint characteristics are discussed in the optimization problem. These are the main constraints of the problem, equating the differences in elevation heads, pressure heads, and head losses through the pipeline sections. Section 8.3.2 states the minimum operation pressure constraint at the irrigation nodes. Minimum and maximum water flow velocities need to be imposed for the network's reliability and these constraints are introduced in Section 8.3.3. The law of mass conservation constraints (water flow balances) discussed in Chapter 5 is presented in Sections 8.3.4 and 8.3.5.

8.2 The objective function in the MINLP form

The objective function for the model was set up in Chapter 7 for maximizing the Net Present Value of the water distribution network investment as:

$$
\begin{aligned}
NPV \quad = \quad & \sum_{ij \, \in \, H} B\left(ij,k\right) \cdot \left[\frac{\left(1+r\right)^{n} - \left(1+e_{f}\right)^{n}}{\left(1+r\right) - \left(1+e_{f}\right)} \cdot \frac{1}{\left(1+r\right)^{n}} \right] \\
& - EC \cdot \left[\frac{\left(1+r\right)^{n} - \left(1+e_{d}\right)^{n}}{\left(1+r\right) - \left(1+e_{d}\right)} \cdot \frac{1}{\left(1+r\right)^{n}} \right] \\
& - \sum_{ij \, \in \, A} PC(i,j) \cdot \left[1 + \left(0.005 + \frac{1}{n} \right) \cdot \frac{\left(1+r\right)^{n} - 1}{r\left(1+r\right)^{n}} \right] \\
& - \widehat{\beta} \cdot PW \cdot \left[1 + \left(0.02 + \frac{1}{n} \right) \cdot \frac{\left(1+r\right)^{n} - 1}{r\left(1+r\right)^{n}} \right] \\
& - \sum_{ij \, \in \, N} IS\left(i,j\right)
\end{aligned}
\tag{8.1}
$$

where $EC = \sum_{s} \Phi \left[Q_{pu}\left(s\right) P_{pu}\left(s\right) \right].$

The objective function concavity

The *first term* in the above definition of the model's objective function is the discounted net-benefits of each hydrant. These costs are modeled as 'fixed' benefits in the form $\sum_{ij \, \in \, H} B\left(i,j\right) = \sum_{ij \, \in \, H} \sum_{k} return \cdot NODE\left(ij,k\right),$ where $return$ is a parameter for the aggregated benefit of the ten farms in each hydrant. $NODE\left(ij,k\right)$ is a binary variable for the selection of the hydrant to be included in the optimal solution. This binary variable simultaneously selects the type of pipeline k connecting the hydrant in the network. All the *first term* components are linear functions, and can be stated as concave for the maximization problem[1].

The *second term* represents the discounted diesel costs (EC) defined as:

$$EC \cdot \left[\frac{(1+r)^n - (1+e_d)^n}{(1+r)-(1+e_d)} \cdot \frac{1}{(1+r)^n} \right] = \Phi \cdot \sum_s Q_{pu}(s)\, P_{pu}(s) \cdot \left[\frac{(1+r)^n - (1+e_d)^n}{(1+r)-(1+e_d)} \cdot \frac{1}{(1+r)^n} \right], \tag{8.2}$$

with $\Phi = \left[\frac{\rho g}{\eta} \cdot \left(T_{sh} \cdot \left(\frac{1}{6} + \frac{1}{10} \right) \cdot 140 \right) \cdot cf \cdot p_d \right]$ as discussed in Chapter 7.

As can be seen, the diesel costs are nonlinearly dependent on a bilinear product between the variables pump's discharge $Q_{pu}(s)$ and delivered head $P_{pu}(s)$ for each shift s. Each of these bilinear products is a convex function. The sum of these bilinear products, $\sum_s Q_{pu}(s)\, P_{pu}(s)$, is also a convex function. The overall discounted diesel costs function is, accordingly, nonlinear convex.

The *third term* represents the investment costs in the pipeline network and discounted O&M and depreciation costs. The investment costs in the pipeline system are modeled with a 'fixed' cost as: $PC(i,j) = \sum_k pipecost(k) \cdot l(ij) \cdot NODE(ij,k)$. In this function, $pipecost(k)$ is a parameter for the unit cost (EGP/m) of a pipeline with diameter type k and lenght $l(ij)$ in m. The term $NODE(ij,k)$ is again the same binary variable for choosing hydrants and respective pipeline diameters. $PC(i,j)$ is a linear function and will be stated here as convex.

The *fourth term* represents the investment and again discounted non-energy O&M and depreciations costs of the pumping unit (see Chapter 7). These costs were modeled as a linear relationship between pump prices and power demanded by the network. In this function only the peak maximal demanded power will be taken for estimating the costs of the unit pump. For example, the pump price is estimated for the highest power demand in all 3 irrigation shifts, i.e. $PW = \max\{PW(1);\ PW(2);\ PW(3)\}$.

[1] Linear functions are automatically both concave and convex.

The power demand function was defined in Chapter 5 and is recalled to here for each shift s as:

$$PW\left(s\right)=\frac{\rho g\cdot\left[Q_{pu}\left(s\right)P_{pu}\left(s\right)\right]}{\eta}\ . \tag{8.3}$$

where $PW\left(s\right)$ is a convex function in the bilinear form $\left[Q_{pu}\left(s\right)P_{pu}\left(s\right)\right]$ as defined above.

The *fifth term* $IS\left(i,j\right)$ represents the aggregated costs of implementing drip and sprinkler irrigation systems at each of the ten farms in an irrigation node. The term is modeled as a 'fixed' cost component $IS\left(i,j\right)=\sum_{k} nodecost\cdot NODE(ij,k)$, where '*nodecost*' is the parameter for the implementation costs of the drip and sprinkler systems. This is a linear function and considered here as convex.

The objective function can now be reformulated as:

$$\begin{aligned}
NPV \quad = \quad & \sum_{ij\in H}\sum_{k} return\cdot NODE\left(ij,k\right)\cdot\left[\frac{\left(1+r\right)^{n}-\left(1+e_{f}\right)^{n}}{\left(1+r\right)-\left(1+e_{f}\right)}\cdot\frac{1}{\left(1+r\right)^{n}}\right] \\
& -\Phi\cdot\sum_{s}Q_{pu}\left(s\right)P_{pu}\left(s\right)\cdot\left[\frac{\left(1+r\right)^{n}-\left(1+e_{d}\right)^{n}}{\left(1+r\right)-\left(1+e_{d}\right)}\cdot\frac{1}{\left(1+r\right)^{n}}\right] \\
& -\sum_{ij\in A}\sum_{k} pipecost(k)\cdot l(ij)\cdot NODE(ij,k)\cdot\left[1+\left(0.005+\frac{1}{n}\right)\cdot\frac{\left(1+r\right)^{n}-1}{r\left(1+r\right)^{n}}\right] \\
& -285\cdot\max.\left\{PW\left(1\right);\ PW\left(2\right);\ PW\left(3\right)\right\}\cdot\left[1+\left(0.02+\frac{1}{n}\right)\cdot\frac{\left(1+r\right)^{n}-1}{r\left(1+r\right)^{n}}\right] \\
& -\sum_{ij\in H}\sum_{k} nodecost\cdot NODE(ij,k)
\end{aligned} \tag{8.4}$$

The objective function of this MINLP formulation is nonlinear in the bilinear form $Q_{pu}\left(s\right)P_{pu}\left(s\right)$ and concave. It is constituted of sums of concave functions and negative convex functions. In nonlinear programs the existence of a concave objective function and a convex set of constraints would guarantee that a local maxima is in fact the global optimum (Hillier and Lieberman, 2005).

8.3 Model's constraints

8.3.1 The law of energy conservation

One of the main constraints to the objective function maximization is the compliance with the law of energy conservation expressed by Equation 5.22. The Equation is modified here to hold for every shift (s) and every section (i, j) in the network. Because the *kinetic head* is normally low relative to the other head losses (Walski, 2001), it was decided to drop this term in Equation 5.22 for the sake of simplicity. Equation 5.22 is also modified to include all the k pipe diameters options and is presented below as the first model's constraint block.

$$P\left(s,i\right) \ - P\left(s,j\right) + z\left(i\right) - z\left(j\right) = \sum_{k} \frac{10.68 \cdot L\left(ij,k\right)}{C\left(k\right)_{hw}^{1.852} \cdot D\left(ij,k\right)^{4.87}} \cdot Q\left(s,ij\right)^{1.852}, \ \forall \ \left(s,ij\right) \tag{8.5}$$

where,

$z\left(i\right)$ and $z\left(j\right)$ are the elevation heights of section (i, j)

$\frac{p\left(s,i\right)}{\rho g} = P\left(s,i\right)$ is the *inlet* pressure head pressures at section i

$\frac{p\left(s,j\right)}{\rho g} = P\left(s,j\right)$ is the *outlet* pressure head pressures at section j

The factor 10.68, is a parameter for units conversion

$L\left(i,j\right)$ and $D\left(ij,k\right)$ are the length and diameter of the used pipe k at section (i, j)

$Q\left(s,ij\right)$ is the discharge of the pipe at section (i, j), and

C_{hw} is the Hazen-Williams pipe roughness parameter.

The Hazen-Williams (HW) equation on the right side of the constraint Equation 8.5, is a nonlinear and convex function (see Price and Ostfeld, 2013, for a discussion on the convexity of the HW equation and innovative linearization procedures). The energy conservation constraints in Equation 8.5 are nonlinear convex functions.

8.3.2 Minimum pressure at the irrigation hydrants

A certain pressure head has to be assured at each node for operating the drip and sprinkler systems in the farms. The condition is assured in the model by the following constraint,

$$P(s, h_o) \geq P_{\min} \ \forall \ s \text{ and } h_o \in H_o, \tag{8.6}$$

where H_o is the set of nodes operating in shift (s). The minimum operation head $P_{\min} = 40$ m must be guaranteed for each hydrant h_o irrigating in shift s (see Appendix G).

8.3.3 Velocity limits for water flow: Reliability constraints.

The discharge $Q(s, ij)$ flowing in shift (s) through section (i, j) of pipeline type k can be defined as,

$$Q(s, ij) = \pi \cdot \left(\frac{D(k)}{2}\right)^2 \cdot \nu(s, ij, k) \tag{8.7}$$

where, $\pi \cdot \left(\frac{D(k)}{2}\right)^2$ is the area of pipeline's cross-section and $\nu(s, ij, k)$ is the water velocity. Given the same flow $Q(s, ij)$, implementing pipes with lower diameter (lower initial investment costs) increases velocity, friction and energy costs. Implementing pipes with higher diameters (higher initial investment costs) implies lower velocities, lower friction and lower energy costs. The increases or decreases in velocity due to lower or higher pipe diameters should be such that velocity is not lower than 1.5 $\text{m} \cdot \text{s}^{-1}$ and not higher than 3 $\text{m} \cdot \text{s}^{-1}$ (Walski, 2001). These are important reliability constraints implemented in the model as:

$$\nu\left(s,ij,k\right)=\frac{Q\left(s,ij\right)}{\pi\cdot\left(\frac{D\left(k\right)}{2}\right)^{2}}\leq 3,\ \forall\ \left(s,ij\right)\ \text{and}\ k \tag{8.8}$$

$$\nu\left(s,ij,k\right)=\frac{Q\left(s,ij\right)}{\pi\cdot\left(\frac{D\left(k\right)}{2}\right)^{2}}\geq 1.5,\ \forall\ \left(s,ij\right)\ \text{and}\ k \tag{8.9}$$

8.3.4 Law of conservation of mass for network's main pipelines

The following constraints refer to the discharge balances across the main pipelines of the network for each shift s, where L is the set containing the sections of the main pipelines, i.e. $L=\{(1,2);(2,7);(7,15);(15,23)\}$ in Figure 8-1. The inflow in each node j of a main line section (i,j) must equal the sum of the outflows from that node.

$$Q\left(s,ij\right)=\sum_{\overline{ij}\ \in\ \overline{L}}Q\left(s,\overline{ij}\right),\ \forall\ s,\ ij\in L \tag{8.10}$$

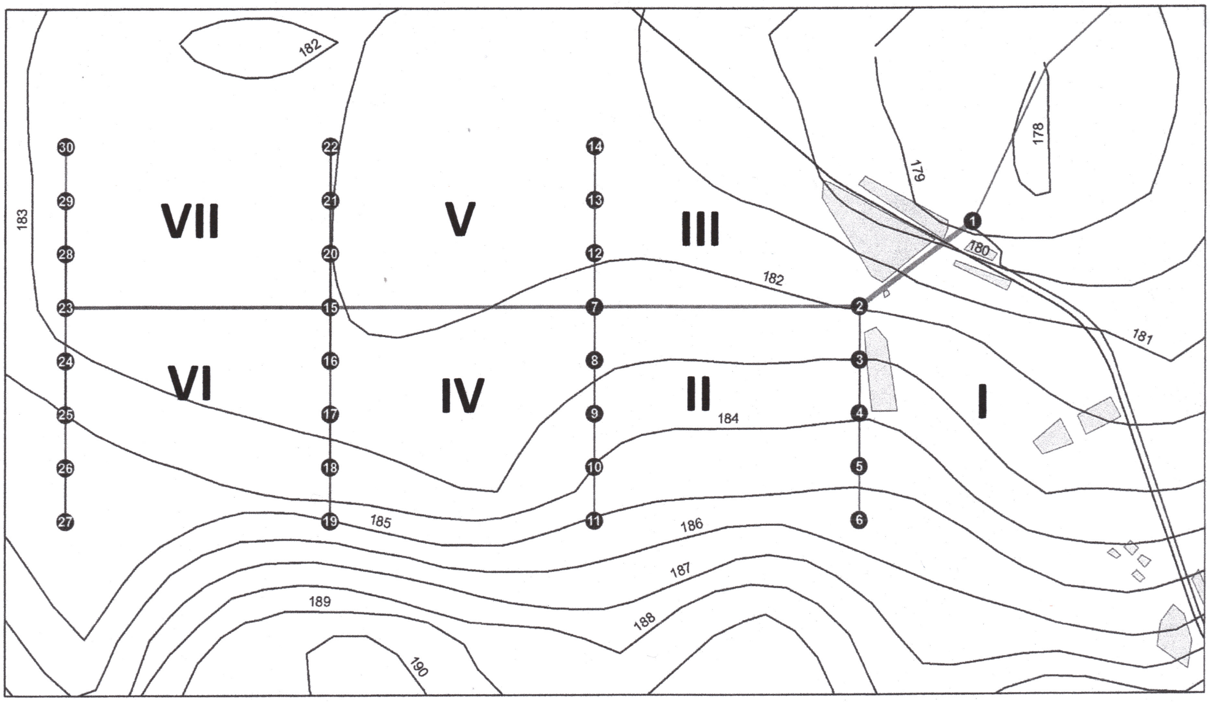
The set $\overline{L}$ contains the other main lines and branch sections dowstream from that same node j.

8.3.5 Law of conservation of mass for network's branches

The conservation of mass in each branch can be explained by taking branch I as example. Branch I contains the sections $\{(2,3);(3,4);(4,5);(5,6)\}$. The subset $\overline{ij}\left(ij\right)$ contains the sections downstream of each single section ij from I. E.g. $\overline{ij}\left(3,4\right)=\{(4,5);(5,6)\}$. The same scheme applies through the other branches $(II-VII)$ in the network, and it is represented in the following equation block.

$$Q\left(s,ij\right)=\sum_{\overline{ij}(ij)}Q\left(s,\overline{ij}\right),\ \forall\ s\ \text{and}\ ij\in(I-VII) \tag{8.11}$$

Figure 8-1: Kalabsha Water Distribution Network



8.4 Summary

This chapter has presented the first operationalization steps for linearizing the MINLP mathematical formulation of the WDN dimensioning original problem. The next chapter will be concerned with the non-linearities in the energy costs of the model $EC = \Phi \cdot \sum_s Q_{pu}(s)\, P_{pu}(s)$, and the nonlinear HW function in the law of energy conservation constraints, i.e:

$$\sum_k \frac{10.68 \cdot L(ij,k)}{C(k)_{hw}^{1.852} \cdot D(ij,k)^{4.87}} \cdot Q(s,ij)^{1.852}, \ \forall \ (s,ij).$$

Chapter 9

An MILP approximation of the WDN dimensioning problem

9.1 Introduction

This chapter introduces the approach used in this dissertation for solving the MINLP type of problem. The approach consists of a Mixed Integer Linear Programming (MILP) approximation of the original MINLP form. The MILP formulation will make use of binary variables for linear approximation of the Hazen-Williams (HW) equation in the constraints and for piece-wise approximation of the pump power and energy cost equations in the objective function.

The chapter is structured as follows: Section 9.2 introduces the methods used in this dissertation to approximate the nonlinearity in the objective function of the model. Section 9.3 presents the operationalization of the law of energy conservation constraint including the linearization of the HW equation. Section 9.4 shows the constraints for the minimum operating pressure on each node. Section 9.5 presents the piecewise linearization of the power and energy cost equation in the objective function of the model. Section 9.6 introduces the pipeline cost equations and Section 9.7 the velocity constraints to water flow (network's reliability constraints). The constraints regarding the law of conservation of mass (discharge balances) in the network are presented in Sections 9.8 and 9.9. Section 9.10 introduces several other necessary constraints for the model's operationalization. The chapter closes with Section 9.11 presenting the reformulated objective function of the model.

9.2 Approximating nonlinear functions

The non-linearities of this model were approximated using the following two standard approaches:

The first approach is suitable when a function $g(x_i, ...x_r)$ can only take a restricted number of values i.e. $x_i \in \{\overline{x}_1, ...\overline{x}_r\}$. In this case the function $g(x_i)$ can be approximated by:

$$\widehat{g}(\overline{x}) = \sum_{i=1}^{r} \lambda_i g(\overline{x}_i) \tag{9.1}$$

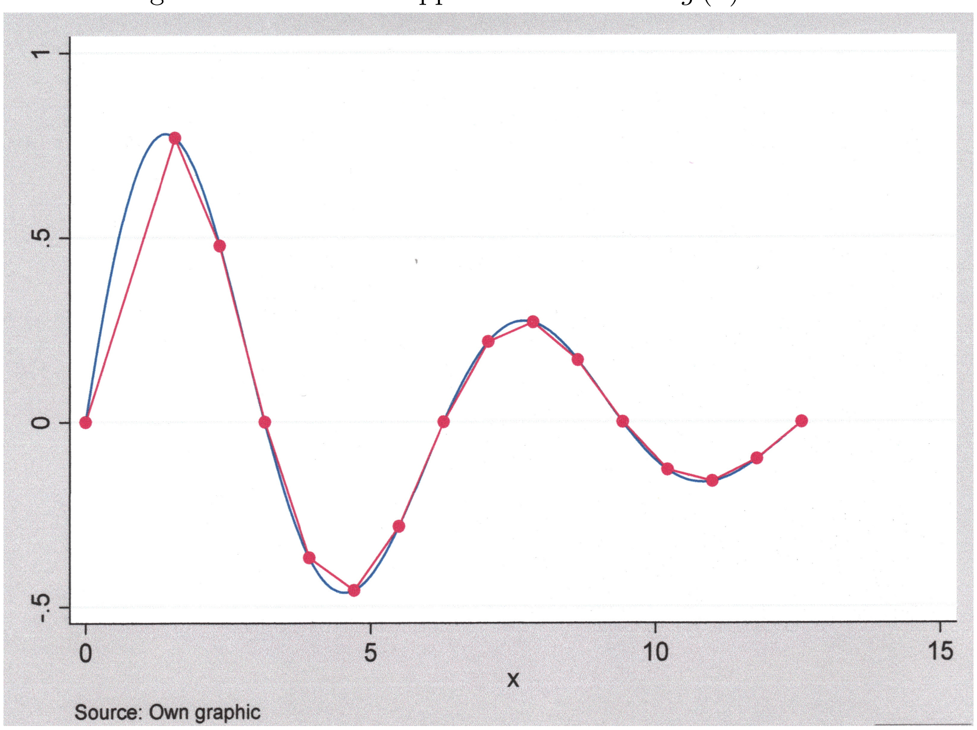
$$\sum_{i=1}^{r} \lambda_i = 1 \tag{9.2}$$

$$\lambda_i = 0 \text{ or } 1,\ i = 1, ..., r \tag{9.3}$$

In the above structure, because the sum of the binary variables λ_i is forced to be one, only one value of $g(\overline{x}_i)$ will be chosen. In this case, we have what is called a special ordered set of type one (SOS1). The approach is suitable when it is requested that exactly one x_i is different from zero (McCarl and Spreen, 1997; Bard, 1998). This approach was used in this dissertation to linearize the convex HW Equation 5.21. This is the right approach for the case at hand because there is only one possible set of discharges in each pipeline section for each shift. The discharge in each pipe section depends on the number of nodes irrigating downstream in the respective shift (Section 9.9). For each section (i, j) the HW equation is evaluated only at these discharge *breakpoints* and is forced to take only one of the feasible discharge values per irrigation shift. The linearization of the convex HW equation using ordered sets of type one (SOS1) is addressed in Section 9.3.1.

The second approach is used when it is not required or wished that the x_i variable takes only fixed predetermined values. The approach is suitable for performing piecewise approximations to nonlinear functions and makes use of special ordered sets of type two (SOS2), i.e. ordered sets that have the restriction that at most two adjacent x_i are non-zero (McCarl and Spreen, 1997; Bard, 1998). The approach consists (as above) on introducing *breakpoint* coordinates $x_1, ...x_n$ along the x axis and evaluates the g function on this domain of the *breakpoints.* The value of the function is approximated by $\widehat{g}(\overline{x})$ as convex combination of $g(x_i)$ and $g(x_{i+1})$. The approximation $\widehat{g}(\overline{x})$ is built upon the linear segments connecting x_i and x_{i+1}, e.g. $x_i \leq \overline{x} \leq x_{i+1}$. A piecewise linear approximation can be seen in Figure 9-1 for the hypothetical function $g(x) = \exp(-x/6) \cdot \sin(x)$.

Figure 9-1: Piecewise approximation of the $g(x)$ function



For the piecewise approximation, the variable λ_i is now defined as continuous (not binary, as in the first approach above) but nevertheless uniquely defined in $[0, 1]$. Any linear combination of the breakpoints x_i can be taken as:

$$\overline{x} = \sum_{i=1}^{r} \lambda_i x_i, \tag{9.4}$$

where r is the number of chosen breakpoints. The approximated value of $g(x_i)$ we are searching is given by,

$$\widehat{g}(\overline{x}) = \sum_{i=1}^{r} \lambda_i g(x_i) \tag{9.5}$$

$$\sum_{i=1}^{r} \lambda_i = 1 \tag{9.6}$$

$$\lambda_i \geq 0, \ \forall \ i \in \{1, ..., r\} \text{ and } \lambda_i \in SOS2 \tag{9.7}$$

The variables λ_i are defined as being of the SOS2 type. It means, no more than two λ_i are positive and adjacent, i.e. λ_i, and λ_{i+1}. The set of SOS2 variables includes the extra following constraints to force any chosen x_i value to be associated with a convex combination of consecutive breakpoints. For the purpose the binary variable δ_i is introduced in association with each one of the breakpoint intervals $[x_i, x_{i+1}]$.

Example 1 *Let us imagine we have five breakpoints and associated* λ_r $(r = 1, ...5)$ *and four intervals* $(n = 1, ..., 4)$ *associated with four binary variables* δ_n. *The approximated value* $\widehat{g}(\overline{x})$ *can be calculated by imposing the following extra conditions in the above structure:*

$$\sum_{i=1}^{4} \delta_i = 1 \tag{9.8}$$

$$\lambda_1 \leq \delta_1 \tag{9.9}$$

$$\lambda_2 \leq \delta_1 + \delta_2 \tag{9.10}$$

$$\lambda_3 \leq \delta_2 + \delta_3 \tag{9.11}$$

$$\lambda_4 \leq \delta_3 + \delta_4 \tag{9.12}$$

$$\lambda_5 \leq \delta_3 \tag{9.13}$$

The first constraint (9.8) imposes that the convex combination $\overline{x} = \sum_{i=1}^{r} \lambda_i x_i$, can only be effective in one single interval $[x_i, x_{i+1}]$. Lets take the example, say $\delta_2 \neq 0$. Then is clear from (9.10) and (9.11), that only the variables λ_2 and λ_3 associated with breakpoints 2 and 3 can be different from zero (the adjacent property of the approximation is assured).

This second approach will be used in Section 9.5 to linearize the power and energy cost functions presented in Chapter 8, Equation 8.2.

9.3 Equation block 1: Law of energy conservation

As can be seen in Figure 8-1 the starting size of the irrigation water distribution network consists of a total of 30 nodes (from which 25 are irrigation hydrants) connected by 29 pipeline sections (i, j). The water discharge q_H at each hydrant when irrigating is constant and determined *a priori* (see Chapter 6 and Appendix G). A discharge $Q(s, ij)$ in shift s and section (i, j) will always be equal to some multiple of q_H dependent on the number of hydrants simultaneously irrigating downstream. For example, assume that only branch I with all hydrant 3, 4, 5, and 6 is irrigating (all hydrants "on"). The discharge of each hydrant is constant and referred as q_H, the discharge in pipe $(1, 2)$ is the same as in $(2, 3)$ and equals $4 \cdot q_H$ because four hydrants are "*on*" downstream. Discharge in Sections $(3, 4)$, $(4, 5)$ and $(5, 6)$ are respectively: $3 \cdot q_H$, $2 \cdot q_H$ and q_H. If a node j is *on* or *off* is determined by the "valve" binary variable $V(s, j)$. According to this, discharges $Q(s, ij)$ in branch I of the network are modeled using the following equation:

$$Q(s, ij) = \sum_{j \in I} V(s, j) \cdot q_H, \quad \forall \; s \text{ and } ij \in I \tag{9.14}$$

The same equation applies for each branch section of the network. Furthermore, these equations will also determine the irrigation schedule for the network, i.e. which hydrants are *on* or *off* in each shift ($V(s, j) = 1$ or $V(s, j) = 0$).

9.3.1 Linearization of the Hazen-Williams equation

The HW Equation 5.21[1] is repeated here for a better understanding of the linearization procedure[2]:

$$h_f = \frac{10.68 \cdot L\left(ij\right)}{C_{hw}^{1.852} \cdot D\left(k\right)^{4.87}} \cdot Q\left(s, ij\right)^{1.852}. \tag{9.15}$$

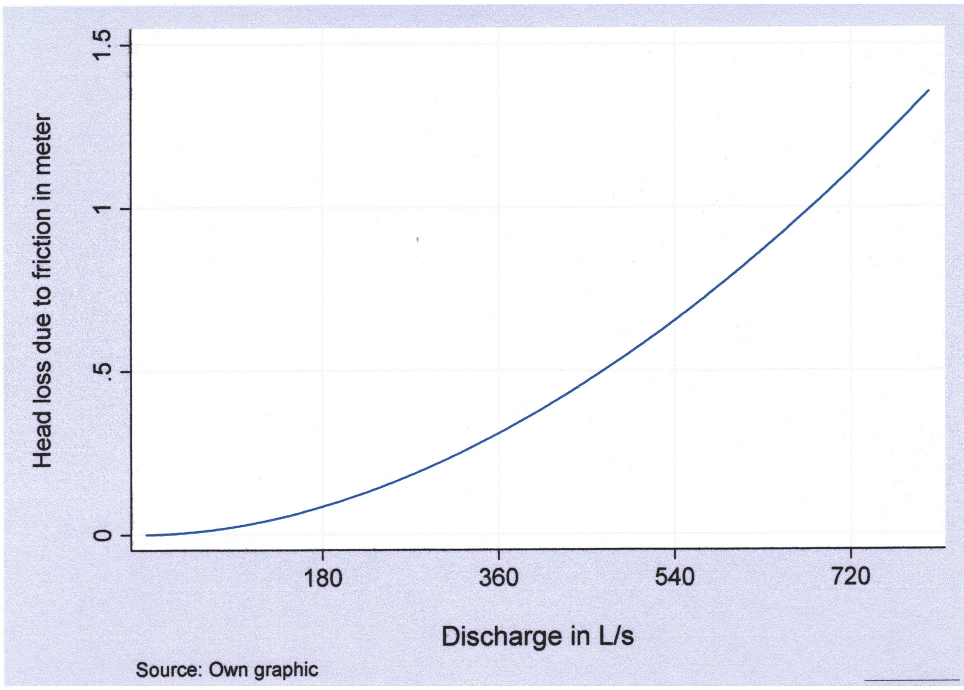
The length of each pipeline $L\left(ij\right)$ is constant and will be represented by the parameter $l\left(ij\right)$. The diameters $D\left(k\right)$ of the different pipeline options for each pipeline section is set as the parameter $diam\left(k\right)$, where k is a set for the available common diameters in the Egyptian market. The only remaining variable in this equation is the discharge $Q\left(s, ij\right)$. The discharge is raised to a nearly quadratic power inducing a convex constraint to the model (see Price and Ostfeld, 2013). In Figure 9-2 one can see a simulation of the values and curvature of the head losses generated by this function for section $(i, j) = (2, 3)$, when assuming a PVC pipeline with $C_{hw} = 150$, diameter $D\left(k\right) = 620\ mm$, and section length $L\left(2, 3\right) = 200\ m$.

The discharge $Q\left(s, 2, 3\right)$ equals the sum of discharges q_H of the hydrants operating downstream of section $(2, 3)$ If only hydrant 3 ($j = 3$) is irrigating, the discharge $Q\left(s, 2, 3\right)$ will be 180 $\mathrm{L \cdot s^{-1}}$ for shift s. If two hydrants are irrigating downstream the discharge will be 360 $\mathrm{L \cdot s^{-1}}$ and so forth. Because the discharge of a pipeline in each section and shift is always a multiple of q_H, the friction losses caused by the different discharges can be modeled using binary variables in a special ordered set $SOS1$ linearizing the HW equation.

The selection of the pipe diameter and discharge in shift s is accomplished by using the binary variable $I\left(ij, s, d, k\right)$. The variable will take the value *one* for section (i, j) selecting in shift s an appropriate discharge d and pipe type k[3]. The variable will take the value *zero*

[1] See also Equation 8.5
[2] See Chapter 5 for the variables and parameter units.
[3] It will be shown that only one pipeline type k is allowed in each (i, j), see Section.9.10.

Figure 9-2: Calculation of head losses using the Hazen-Williams equation



in the shift if no water is flowing in the section. If the variable remains zero for all shifts, the hydrant is dropped from the optimal solution. The higher the discharge, the higher will be the friction losses in the pipe. The higher the diameter of pipe type (k) the lower the friction losses will be. This fact expresses a trade-off between the initial investment costs in the pipelines and energy (head loss) costs. The higher the pipeline's diameters the higher the initial investment costs but the lower will be the energy costs (lower head losses).

The chosen discharge for each shift and section in Equation 9.14 is set to be equal to:

$$Q\left(s,ij\right)=\sum_{d}\sum_{k}\left(val\left(d\right)\,\cdot q_{H}\right)\,\cdot I\left(ij,s,d,k\right),\;\forall\;\left(s,ij\right), \tag{9.16}$$

where $val\left(d\right)\in\{1,2,3,...\}$ is a parameter for building multiples of q_H. Furthermore, because variable $V\left(s,j\right)$ sets the irrigation hydrants "on" or "off " (9.14) must equal (9.16) for every branch (i,j) giving:

$$Q\left(s,ij\right)=\sum_{j\,\in\,\subset\,I}V(s,j)\cdot q_{N}=\sum_{d}\sum_{k}\left(val\left(d\right)\,\cdot q_{H}\right)\,\cdot I\left(ij,s,d,k\right),\;\forall\;s\text{ and }ij\in I. \tag{9.17}$$

Substituting (9.16) in Equation (9.15), we get the linearized HW Equation below, where the indicator variable $I\left(ij,s,d,k\right)$ will choose the appropriate pipe diameter k, discharge and with it the head loss for each shift and section $\left(i,j\right)$. The lenght of each pipeline section $L\left(ij,k\right)$ is constant and can be substituted by the parameter $l(ij)$. The diameter variable $D\left(ij,k\right)$ in Equation 8.5 can now be considered as a parameter as well, $diam\left(k\right)$, controlled by the binary variable $I\left(ij,s,d,k\right)$.

$$hf=\sum_{d}\sum_{k}l(ij)\cdot 10.68\cdot diam\left(k\right)^{-4.87}\cdot\left(\frac{val\left(d\right)\,\cdot q_{H}}{C_{hw}\left(k\right)}\right)^{1.852}\cdot I\left(ij,s,d,k\right),\;\forall\;\left(s,ij\right) \tag{9.18}$$

Given the linearized HW Equation (9.18), the energy conservation Equation 8.5 can be made operational by[4]:

$$P\left(s,i\right)-P\left(s,j\right)+z(i)-z(j)= \tag{9.19}$$

$$\sum_{d}\sum_{k}10.68\cdot l(ij)\cdot diam\left(k\right)^{-4.87}\cdot\left(\frac{val\left(d\right)\,\cdot q_{H}}{C_{hw}\left(k\right)}\right)^{1.852}\cdot I\left(ij,s,d,k\right),\;\forall\;\left(s,ij\right)$$

[4] The hydrants discharge q_H is given here in $\mathrm{m}^3\cdot\mathrm{s}^{-1}$.

9.4 Equation block 2: The minimum demanded pressure at the irrigating hydrants

The minimum pressure demanded at the irrigation hydrants is given in Equation (9.20) with $P_{\min} = 40$ m as:

$$P(s, h_o) \geq 40 \cdot V(s, h_o) \;\; \forall \; s \text{ and } h_o \in H. \tag{9.20}$$

where H is the set of irrigating hydrants in shift s.

9.5 Equation block 3: The energy cost equation

The energy cost in Equation 8.2 is a nonlinear and convex function in the bilinear form $Q \cdot P$, the discharge and pressure delivered by the system's pump in every shift s. Bilinear forms are non-separable and not possible to linearize directly. A function of the form $f(x) = x_1^2 + x_2^2 + x$, is separable because each one of the terms x_1^2, x_2^2, and x are a function of only one variable. The function could be linearized through the sum of piecewise approximations of the single terms x_1^2 and x_2^2. Bilinear forms are able to be converted into a separable forms, which can be afterwards linearized using piecewise approximations (McCarl and Spreen, 1997; Bard, 1998; Williams 2013).

Linearization of energy cost equation in the objective function

The given bilinear form $Q \cdot P$, can be transformed in two separable quadratic functions:

$$Q \cdot P = \left(\frac{Q + P}{2}\right)^2 - \left(\frac{Q - P}{2}\right)^2 \tag{9.21}$$

where for this dissertation Q and P are standardized in a first step by: $Q = \dfrac{Q(s;1,2)}{maxQ}$, and $P = \dfrac{P(s,1)}{maxP}$. Where $maxQ$ is the maximal range of discharges, and $maxP$ the maximal range of demanded head from the pumping system. The variables are now defined in the domain $[0,1]$[5]. For clarity, one defines

$$Y_1 = \left(\frac{Q+P}{2}\right) \text{ and } Y_2 = \left(\frac{Q-P}{2}\right) \ , \tag{9.22}$$

with domains $0 \le Y_1 \le 1$ and $-\frac{1}{2} \le Y_2 \le \frac{1}{2}$.

The bilinear form is converted to the difference of two separable nonlinear functions given by:

$$Q \cdot P = Y_1^2 - Y_2^2 \tag{9.23}$$

The procedure now follows the *second approach* discussed in Section 9.2 for approximating the quadratic functions Y_1^2 and Y_2^2.

Remembering that,

$$Y_1\left(s\right) = \frac{Q\left(s;\ 1,2\right)}{2 \cdot \max Q} + \frac{P\left(s,1\right)}{2 \cdot \max P} \tag{9.24}$$

$$Y_2\left(s\right) = \frac{Q\left(s;\ 1,2\right)}{2 \cdot \max Q} - \frac{P\left(s,1\right)}{2 \cdot \max P} \tag{9.25}$$

the variables will be approximated by using the breakpoint parameters $pointsa\left(a\right)$ and $pointsb\left(b\right)$ and the weight variables $A\left(s,a\right)$ and $B\left(s,b\right)$ framed in a Special Ordered Set of type 2 (SOS2)[6].

$$Y_1\left(s\right) = \sum_a A\left(s,a\right) \cdot pointsa\left(a\right),\ \forall\ s \tag{9.26}$$

[5] The discharge $Q\left(s,1,2\right)$ is given for the pump's section $\left(i,j\right) = \left(1,2\right)$ in $\mathrm{m}^3\ \mathrm{s}^{-1}$and $P\left(s,1\right)$ is the pump's pressure in node 1 given in m

[6] Variables $A\left(s,a\right)$ and $B\left(s,b\right)$ are framed in a Special Ordered Set $SOS2$, which means that at most only two adjacent $A\left(s,a\right)$, and $A\left(s,a+1\right)$ are non-zero. The same applies for the $B\left(s,b\right)$ variables. The use of this variable structure leads to simplified and efficient computations when doing piece-wise approximation of non-linear curves (Bard, 1998; Brooke et al., 2005)

$$Y_2(s) = \sum_b B(s,b) \cdot pointsb(b), \; \forall \; s \tag{9.27}$$

The parameter $pointsa(a) \in \{0,\ 0.125,\ 0.250,\ 0.375,\ 0.5,\ 0.625,\ 0.75,\ 0.875,\ 1\}$ represent 9 breakpoints for approximation in the domain $0 \leq Y_1 \leq 1$ of Y_1, and the parameter $pointsb(b) \in \{-0.5,\ -0.375,\ -0.25,\ -0.125,\ 0,\ 0.125,\ 0.25,\ 0.375,\ 0.5\}$ are breakpoints in the domain $-\frac{1}{2} \leq Y_2 \leq \frac{1}{2}$ of Y_2.

$$\sum_a A(s,a) = 1, \; \forall \; s \tag{9.28}$$

$$\sum_b B(s,b) = 1, \; \forall \; s \tag{9.29}$$

and finally:

$$Y_1^2(s) = \sum_a A(s,a) \cdot (pointsa(a))^2, \; \forall \; s \tag{9.30}$$

$$Y_2^2(s) = \sum_b B(s,b) \cdot (pointsb(b))^2, \; \forall \; s \tag{9.31}$$

Let the variable $\overline{QP}(s)$ represent the linear approximation to the product $Q \cdot P$ as:

$$\overline{QP}(s) = \sum_a A(s,a) \cdot (pointsa(a))^2 - \sum_b B(s,b) \cdot (pointsb(b))^2, \; \forall \; s$$

From the above discussion, the author now derives the following relationships. If

$$\frac{Q(s,\ 1,2)}{maxQ} \cdot \frac{P(s,1)}{maxP} = \overline{QP}(s), \tag{9.32}$$

which is equivalent to say,

$$Q(s,\ 1,2) \cdot P(s,1) = \overline{QP}(s) \cdot maxQ \cdot maxP, \; \forall \; s \tag{9.33}$$

then the power function in Equation (8.3) can be finally linearized as,

$$PW\left(s\right)=\frac{\rho g\cdot\ \left[\overline{QP}\left(s\right)\ \cdot maxQ\cdot maxP\right]}{\eta}\ \text{in}\ \left[\text{kW}\right],\ \forall\ s \tag{9.34}$$

The energy cost (EC) is calculated by multiplying the consumed power from the above equation with the number of operation hours a year $\left(Thrs\right)$, and with the price of diesel p_d given in $EGP\cdot\text{L}^{-1}$.

$$EC=\sum_{s}\Phi\cdot\ \left[\overline{QP}\left(s\right)\ \cdot maxQ\cdot maxP\right] \tag{9.35}$$

where $\Phi=\left[\frac{\rho g}{\eta}\cdot T_{sh}\cdot\left(\frac{1}{6}+\frac{1}{10}\right)\cdot 140\cdot cf\cdot p_d\right]$, as defined in Chapter 7.

9.6 Equation block 4: Pipeline costs

The pipeline cost function was defined in the objective function in Chapter 8 as:

$$\sum_{ij\in A}PC\left(i,j\right)=\sum_{ij}\sum_{k}pipecost\left(k\right)\ \cdot l\left(ij\right)\ \cdot NODE(ij,k) \tag{9.36}$$

9.7 Equation block 5: Velocity constraints for water flow

Reliability constraints are introduced in the model through lower and upper bounds for water flow velocity. The velocity constraints imposed in Equations (8.9) and (8.8) are also made operational by using the indicator variable $I\left(ij,s,d,k\right)$ according to:

$$\sum_{d}\sum_{k}\left(\frac{val\left(d\right)\cdot q_H}{\pi\cdot\left(\frac{diam(k)}{2}\right)^2}\cdot I\left(ij,s,d,k\right)\right)\leq 3\ \forall\ s \text{ and } (i,j) \tag{9.37}$$

$$\sum_{d}\sum_{k}\left(\frac{val\left(d\right)\cdot q_H}{\pi\cdot\left(\frac{diam(k)}{2}\right)^2}\cdot I\left(ij,s,d,k\right)\right)\geq 1.5\cdot\sum_{d}\sum_{k}I(ij,s,d,k)\ \forall\ s \text{ and } (i,j) \tag{9.38}$$

On the right hand side of Equation 9.38, the velocity lower bound 1.5 (m s^{-1}) is multiplied by $\sum_d\sum_k I\left(ij,s,d,k\right)$ for assuring that Equation 9.38 still represents a feasible constraint, when no water is flowing in the section in a certain irrigation shift, i.e. when $\sum_d I\left(ij,s,d,k\right)=0\ \forall\ s$ and (i,j).

9.8 Equation block 6: Law of conservation of mass for network's main pipelines

The discharge balances for the main pipelines indicated in general Equation (8.10) will unfold as:

$$Q\left(s,\text{“1”},\text{“2”}\right)=Q\left(s,\text{“2”},\text{“3”}\right)+Q\left(s,\text{“2”},\text{“7”}\right) \tag{9.39}$$

$$Q(s,\text{“2”},\text{“7”})=Q(s,\text{“7”},\text{“12”})+Q(s,\text{“7”},\text{“8”})+Q(s,\text{“7”},\text{“15”}) \tag{9.40}$$

$$Q(s,\text{“7”},\text{“15”})=Q(s,\text{“15”},\text{“20”})+Q(s,\text{“15”},\text{“16”})+Q(s,\text{“15”},\text{“23”}) \tag{9.41}$$

$$Q(s,\text{“15”},\text{“23”})=Q(s,\text{“23”},\text{“24”})+Q(s,\text{“23”},\text{“28”}) \tag{9.42}$$

9.9 Equation block 7: Law of conservation of mass for network's branches

The balance constrains in the following block are modeled using the binary variables $V\left(s, \overline{j}\right)$. These variables indicate if a certain hydrant is "on" or "off", i.e. irrigating or not irrigating. The parameter q_H is again the discharge of each single hydrant when operating. Multiples of q_H can be constructed by summing the binary variables $V(s, \text{“}j\text{”})$ which are "on". The sum of the binary variables gives the whole $Q\,(i, j)$ discharge. The Equations for the discharge balances in the different branches of Figure 8-1 are expressed by Equations 8.11 to **??** from Chapter 8. These will unfold according to the branches I to VII, where the set $\overline{ij}\,(ij)$ always indicates the pipeline sections downstream from (i, j):

$$Q\,(s, ij) = \sum_{\overline{ij}(ij)} V(s, \overline{j}) \cdot q_H, \quad \forall\ s \text{ and } ij \in I \tag{9.43}$$

$$Q\,(s, ij) = \sum_{\overline{ij}(ij)} V(s, \overline{j}) \cdot q_H, \quad \forall\ s \text{ and } ij \in II \tag{9.44}$$

$$Q\,(s, ij) = \sum_{\overline{ij}(ij)} V(s, \overline{j}) \cdot q_H, \quad \forall\ s \text{ and } ij \in III \tag{9.45}$$

$$Q\,(s, ij) = \sum_{\overline{ij}(ij)} V(s, \overline{j}) \cdot q_H, \quad \forall\ s \text{ and } ij \in IV \tag{9.46}$$

$$Q\,(s, ij) = \sum_{\overline{ij}(ij)} V(s, \overline{j}) \cdot q_H, \quad \forall\ s \text{ and } ij \in V \tag{9.47}$$

$$Q\,(s, ij) = \sum_{\overline{ij}(ij)} V(s, \overline{j}) \cdot q_H, \quad \forall\ s \text{ and } ij \in VI \tag{9.48}$$

$$Q\,(s, ij) = \sum_{\overline{ij}(ij)} V(s, \overline{j}) \cdot q_H, \quad \forall\ s \text{ and } ij \in VII \tag{9.49}$$

9.10 Other constraints and model equations

Discharges in the pipelines

The variable $I\,(ij, s, d, k)$ controls the discharge in each pipeline section and pipeline type k. The variable can indicate a discharge d only if the node of the section exists (if there is investment), i.e. if the variable $NODE(ij, k)$ equals *one*. This condition is given by the constraint:

$$\sum_d I(ij, s, d, k) \leq NODE(ij, k),\ \forall\ ij,\ \ s,\ k. \tag{9.50}$$

The constraint furthermore states that for each shift only one discharge d is possible for the section.

Number of pipeline types per section

Only one pipe type (k) is allowed per section (i, j), this is assured by the constraint:

$$\sum_k NODE(ij, k) \leq 1\ \forall\ (i, j) \tag{9.51}$$

The connection of the binary variables $NODE(ij, k)$ and $I\,(ij, s, d, k)$ allows the unambiguous determination of the optimal pipe type (k) for each Section. The pipe type k chosen by $I\,(ij, s, d, k)$, will be the same as the one chosen by $NODE(ij, k)$.

Number of times a hydrant irrigates

The constraint below implies that, if a section (i, j) and respective hydrant are to be implemented, i.e. $NODE(ij, k) = 1$, the respective hydrant must irrigate in some shift s, i.e. the variable $V(s, j)$ needs to be one in some shift.

$$\sum_k NODE(ij, k) = \sum_s V(s, j),\ \forall\ s,\ ij,\ \text{and}\ j \notin \{2, 7, 15, 23\}\,. \tag{9.52}$$

The pump unit is priced using the highest demanded power

The pump price was derived through a linear relationship between the power demanded from the system and the market price of centrifugal pumps. The pump unit cost is estimated using the highest demanded power in all the irrigation shifts. Because the operating regime of the pump in this model is flexible, a procedure was implemented for isolating the highest demanded power. The highest power demand is forced to be in the first shift, $PW(‘1’)$, by implementing the following constraints:

$$PW(‘1’) \geq PW(‘2’) \tag{9.53}$$

$$PW(‘2’) \geq PW(‘3’) \tag{9.54}$$

9.11 The objective function in the MILP form

The different components of the objective function were discussed comprehensively in Chapter 8, Section 8.2 and further in the sections of this chapter. The final form is given as:

$$\begin{aligned} NPV \;=\; & \sum_{ij\,\in\,H}\sum_{k} return \cdot NODE\,(ij,k) \cdot \left[\frac{(1+r)^n-(1+e_f)^n}{(1+r)-(1+e_f)} \cdot \frac{1}{(1+r)^n}\right] \\ & -\sum_{s} \Phi \cdot \left[\overline{QP}\,(s) \cdot maxQ \cdot maxP\right] \cdot \left[\frac{(1+r)^n-(1+e_d)^n}{(1+r)-(1+e_d)} \cdot \frac{1}{(1+r)^n}\right] \\ & -\sum_{ij\,\in\,A}\sum_{k} pipecost(k) \cdot l(ij) \cdot NODE(ij,k) \cdot \left[1+\left(0.005+\frac{1}{n}\right) \cdot \frac{(1+r)^n-1}{r\,(1+r)^n}\right] \\ & -285 \cdot PW\,(”1”) \cdot \left[1+\left(0.02+\frac{1}{n}\right) \cdot \frac{(1+r)^n-1}{r\,(1+r)^n}\right] \\ & -\sum_{ij\,\in\,H}\sum_{k} nodecost \cdot NODE(ij,k) \end{aligned} \tag{9.55}$$

Chapter 10

Solving MILP optimization models

10.1 Introduction

This chapter discusses the methods applied in this dissertation for solving the MILP formulation of the WDN dimensioning problem. In general, Integer Programs (IP) can be understood as combinatorial problems, in the sense that the searched optimal solution is some subset of a finite set of all possible feasible combinations of variables values (Wolsey, 1998). If the set of decision variables is finite, one might think that complete enumeration would solve the problem (choosing the variables values combination that maximizes or minimizes the objective function). This is, unfortunately, not easy because, even for a relatively small problem, the total number of possible value combinations grows exponentially with the number of variables. In the case of a pure binary integer program (BIP) considering n variables, the total number of possible combinations will be 2^n, for example for $n = 30$ there would be more than $1 \cdot 10^9$ possible solutions, i.e. different variable values combinations (Schrijver, 1986; Bard, 1998; Wolsey, 1998; Nemhauser and Wolsey, 1999).

Practical problems have hundreds or thousands of variables and enumerating all possible combinations would not be efficient or even feasible. However, there are modern sophisticated algorithms for solving IP problems, such as the *Branch-and-Bound* (BB) algorithm, that are based on *implicit enumerations.* Implicit means here that the procedure can in fact effectively discard, or fathom, a huge amount of unnecessary possible combinations through several feasibility tests and bounds, without having to address them all explicitly (Bard, 1998; Wolsey, 1998; Nemhauser and Wolsey, 1999; Hillier and Lieberman, 2005). Based on these principles, several combinations of methods have appeared, such as the *Branch-and-Cut* algorithm, improving the efficiency of implicit enumeration. These algorithms use (*inter alia*) combinations of cutting planes and selective enumeration through Branch-and-Bound procedures (Bard, 1998; Wolsey, 1998; Nemhauser and Wolsey, 1999). The present MILP optimization model uses the CPLEX 12 solver. The CPLEX 12 uses among others a *Branch and Cut* algorithm which applies *LP relaxations* that divide the original problem into LP sub-problems solved with the simplex algorithm (ILOG, Inc., 2013). The CPLEX 12 solver is capable of handling IP problems with a very large number of integer variables. Hillier and Lieberman (2005) report models with more than 100,000 variables.

10.2 The Branch-and-Cut approach

The *Branch-and-Cut* approach is based on combinations of three types of procedures that, when used together, can significantly reduce the computation time of the solution search. These procedures are: i) an automatic problem pre-processing; ii) cutting plane generation; and iii) application of the *Branch-and-Bound* algorithm (Bard, 1998; Wolsey, 1998; Nemhauser and Wolsey, 1999; Hillier and Lieberman, 2005).

Automatic preprocessing

The automatic pre-processing of the problem seeks to find reformulations of the given problem structure that induce a faster search for the optimal solution. According to Hillier and Lieberman (2005), these reformulations can be performed by: fixing variables, eliminating redundant constraints and tightening constraints[1].

[1] *The reader is referred to Hillier and Lieberman (2005, p. 515) for more details.*

Cutting plane generation

The cutting plane generation introduces appropriate additional linear constraints to the problem, tightening in this way the LP relaxation, i.e. it successively shortens the feasible region of the LP relaxation problem. The aim is to produce an optimal solution to the relaxation problem with integer-valued variables. The additional cuts tighten the feasible region without discarding any possible integer solution. The aim is to generate as many cuts as possible before an efficient *Branch-and-Bound* (BB) algorithm is applied to the solution space left (Hillier and Lieberman, 2005).

Application of the *Branch-and-Bound algorithm*

The *Branch-and-Bound* (BB) algorithm is based on an 'intelligent' enumeration of feasible integer solutions to the problem. The BB procedure is structured in an efficient way, reducing the analysis to a relatively small portion of all possible feasible solutions. The main strategy of the BB algorithm is to divide (branch) the original problem into smaller and smaller sub-problems until an integer solution is found. The BB can be described by the steps: i) branching, ii) bounding and iii) fathoming (Bard, 1998; Wolsey, 1998; Nemhauser and Wolsey, 1999; Hillier and Lieberman, 2005). These different algorithm steps will be discussed in the next sections.

10.3 Solving IP problems with the Branch-and-Bound algorithm

Solving pure binary integer problems (BIP)

In the branching step, the algorithm first applies the LP relaxation to the original problem, say a BIP, substituting the integer constraints by conditions of the type $0 \leq x_j \leq 1$. After a branching variable is selected (say x_1), two branches (smaller sub-problems) are constructed around fixed values of the branching variable x_1, i.e. a *sub-problem 1* for

$x_1 = 0$, and a *sub-problem 2* for $x_1 = 1$.

In the bounding step each of these sub-problems is solved by again applying the LP relaxation and the simplex algorithm. The solutions of these LP relaxation problems represent **bounds** for the searched integer solution.

In the fathoming step, the solutions of the sub-problems are analyzed to see if further branching is necessary, or if the branch can be disregarded from further branching steps. The analysis criteria of the fathoming step are: i) if the LP relaxation bound Z is lower than another calculated bound from some other branch $Z \leq Z^*$, then this branch can be disregarded, i.e. the sub-problem is fathomed; ii) if the LP relaxation solution is unfeasible, the branch is also fathomed; iii) if the LP relaxation already delivers an integer solution, and if this solution is better than the current best solution of another branch (incumbent Z^*), this solution is now the new incumbent, and the new solution is recorded.

If one of these fathom criteria can be proved in any branch, the branch is considered fathomed, i.e. there is no need for further branching out of this sub-problem. If the fathom criteria are not confirmed, the subproblems can be further branched following the enunciated steps: i) branching, ii) bounding, and again iii) fathoming. If there are no remaining unfathomed sub-problems the last found incumbent Z^* is the best bound, i.e. the optimal solution of the IP problem (Bard, 1998; Wolsey, 1998; Nemhauser and Wolsey, 1999).

Solving mixed integer problems (MILP)

The procedure for solving MILP programs is similar to the one for solving pure BIP, as discussed before. In MILP programs we have integers and continuous variables. The BB procedure is applied only to the group of integer variables of the program. Nevertheless, a significant difference from the BB for the BIP exists: a branching variable is also selected for splitting into two sub-problems, but the values attributed to the branching variable in each sub-problem are constructed differently. In the BIP case the variables were binary, and the values for the sub-problems were $x_1 = 0$ and $x_1 = 1$. In the MIP case, the branch variable can take a very large number of integer values (not only 0 or 1) and it would not be feasible to fix the branching variable at all these different integer values. Instead, the sub-problems are created by specifying two ranges of values for the branching variable according to the following procedure: Imagine the LP relaxation applied in a first step to the original problem and a branching variable selected from the LP relaxation showing a fractional value, say x_F. If now $x_{I_{low}}$ can be defined as the highest integer not exceeding x_F the continuous region $x_{I_{low}} < x_F < x_{I_{low}} + 1$ can be excluded from the original solution space, and with it all the fractional solutions belonging in this interval. In this way, the original problem can be branched (split) into two sub-problems around the LP solution x_F imposing the restrictions $x_I \leq x_{I_{low}}$ and $x_I \geq x_{I_{low}} + 1$ on each of the subproblems. The procedure is now identical to the BIP type of problem, i.e. following the steps: i) branching, ii) bounding and iii) fathoming (McCarl and Spreen, 1997; Bard, 1998; Wolsey, 1998; Nemhauser and Wolsey, 1999; Hillier and Lieberman, 2005;).

In the above discussion only a general description of the BB algorithm was presented. However, there are important decisions to be taken during processing that have an impact on the algorithm's efficiency. One important point is the criteria for choosing between possible branching variables. Following Bard (1998) and Taha (1981), the branching variable at each node should be chosen as the one showing the largest fraction value at that node. The assumption is that such a variable would imply the largest change in the objective function and this could bring up more efficient paths of search. Different branching variables can lead to completely different solution paths and overall efficiency. According to Bard (1998) and Taha (1981) the major difficulties when applying BB algorithms are:

1. Proper selection of the next node to be examined
2. Proper selection of the branching variable
3. Complete solution of a continuous problem at each node

The procedures for finding the best node and respective branching variable (points 1 and 2 above) have a major impact on the number of sub-problems to be solved before optimality can be achieved. The third point refers to the "amount" of computations necessary to *completely* solve the LP relaxation at each node, when the only interesting information for deciding whether to drop or keep a node is the associated value of the objective function.

For the proper node selection, Bard (1998) refers to the state-of-the-art available options: (i) *a priori* rules that determine the order in which the tree of nodes will be developed; (ii) adaptive rules that select the node using information such as bounds or function values associated with the live nodes[2]. The most commonly used rules referred to by Bard (1998) are the depth-first plus backtracking search and the breath-first search (for more details on the BB algorithm see: Bard, 1998; Wolsey, 1998, and Nemhauser and Wolsey, 1999).

10.4 The LP relaxation

The term 'relaxation' means (as discussed above) that the original IP is first solved as a linear problem by deleting the integer constraints imposed on the x vector of binary variables. For the pure binary integer case (BIP), these constraints would be $x_j = 0$, or $x_j = 1$, substituted by the relaxation constraints $0 \leq x_j \leq 1$ (McCarl and Spreen, 1997; Bard, 1998; Wolsey, 1998; Nemhauser and Wolsey, 1999).

Applying the LP relaxation means solving a linear problem maximizing or minimizing an objective function $f(x)$ subject to the linear constraints $g(x)_i$, where f and g have linear functional forms, the x_j are the decision variables and b_i is the available amount of resource i. The model can be further detailed as,

[2] Live nodes are those that can still be branched and are not fathomed.

$$Z = \sum_{j=1}^{n} c_j x_j \tag{10.1}$$

subject to the constraints,

$$\sum_{j=1}^{n} a_{ij} x_j \leq b_i, \text{ for } i = 1, ...m - n \tag{10.2}$$

$$x_j \leq 1, \text{ for } j = 1, ...n \tag{10.3}$$

$$x_j \geq 0, \text{for } j = 1, ..., n.$$

The coefficients c_j could be prices or gross margins per unit of the decision variable, i.e. the unit value that the x_j variable takes in the objective function. The coefficients a_{ij} are the quantities of resource i necessary to produce one unit of activity x_j. The set of equations (10.3) represent the relaxation conditions substituting the integer constraints.

The problem can be expressed in matrix notation as,

$$Z = cx \rightarrow \max. \tag{10.4}$$

$$Ax \leq b \tag{10.5}$$

$$x \geq 0 \tag{10.6}$$

where Z is a scalar, the profit or net returns of an investment (in the maximization problem), c is a known $(1 \times n)$ parameter vector (e.g. gross margins), x is a $(n \times 1)$ vector of unknown decision variables, A is a known $(m \times n)$ matrix containing the resource demands coefficients a_{ij}, also now including the coefficients of the $x \leq 1$ relaxation. The b vector is a $(m \times 1)$ known column of constants (e.g. resource availability, endowments, and the 1's from the LP relaxation). The last constraints $x \geq 0$ are the n non-negativity conditions.

The system (10.5) and the non-negativity conditions (10.6) form a whole system of $m+n$ constraint boundary Equations, spanning the feasible n-dimensional solution space. For the **algebraic** identification of the boundary of the feasible region, the original formulation as in (10.5-10.6) needs to be reformulated in the equality general form:

$$a_{i1}x_1 + a_{i2}x_2 + ... + a_{in}x_n = b_i \text{ for all constraints } i \tag{10.7}$$

Each constraint in the equality form (10.7) defines a hyperplane (n is larger than 3 dimensions) representing the boundary of the corresponding constraint. Geometrically, in a n-dimensional space, the interception of n-hyperplanes forms a feasible boundary corner point. The connection between two corner points in the solution space is an edge of the solution space formed by the interception of $(n-1)$-hyperplanes. The whole solution space S is a polyhedron with corner points and edges formed by interceptions of hyperplanes that can be expressed in the algebraic form of Equations 10.7. It can be shown, that if S is bounded and not empty, then it represents a convex polyhedron of solutions, and the optimum solution of a maximization problem is to be found at the corner points of this convex polyhedron. This means that only the corner points (extreme points) need to be analyzed to find the optimum solution (Dantzig, 1997, 2003).

Algebraically, the corner point solutions can be found by simultaneously solving the different possible combinations of systems of n Equations to n unknowns from the $m+n$ whole set of constraint boundary equations (m functional constraints and the n non-negativity conditions). A procedure for finding the optimal solution could consist of solving all possible $n \times n$ systems of equations from the whole set of $m+n$ equations. The total number of possible systems of Equations and total number of possible corner points would be $\binom{m+n}{n} = \frac{(m+n)!}{m!n!}$.

For the majority of real-life optimization problems this would be a highly inefficient procedure, given the potentially immense number of possible corner points in the solution space. The most common procedure for solving linear problems is the simplex algorithm proposed originally by Dantzig (1966).

The simplex algorithm

The simplex algorithm sets the search for the optimal corner point of the solution space along an efficient path, not needing to enumerate and evaluate all the corner points of the feasible solution polyhedron. The simplex algorithm starts the iteration procedure at the origin of the solution space (at the vector null), also called the initial *basic* solution. Moving to other corner points implies changing the variables in the basic solution, i.e. including new variables not originally in the solution, *non-basic* variables. The simplex moves from the null vector point to another adjacent corner point along the edge of the polyhedron connecting them. The new direction in which the simplex should progress, or which of the n possible corner points it should move towards, depends on the contribution of each candidate *non-basic* variable to the objective function. The *non-basic* variable selected to enter the *basic solution*, substituting a basic-variable and determining the next corner point, must provide the largest marginal contribution to the objective function. After the variable is selected a new *basis* is set and a system of n equations to n unknowns is solved, determining the coordinates of the new corner point (possible optimum). The simplex will continue with this iteration procedure until no more *non-basic* variables exist that can increase the objective of the program (Dantzig, 1997, 2003).

The MILP of the present dissertation is solved by applying the CPLEX solver through the GAMS interface (Brooke et al., 2005). The CPLEX solver applies the *Branch-and-Cut* algorithm for solving MILP models among other sophisticated methods.

Chapter 11

Criteria for investment selection and model scenarios

11.1 Introduction

It is assumed in this work that the agency implementing the WDN uses governmental funds that are borrowed in international capital markets to prevailing conditions. If the government is using foreign capital at the prevailing interest rate r, it is then assumed that the returns on investment should be at least as large as the capital cost involved. Following the internal rate of return (IRR) definition, public projects are considered worthwhile if they indicate a positive NPV when discounted at this rate. The capital cost of tapping foreign markets is the minimum acceptable IRR of the investment (Brent, 2000). This chapter introduces the IRR as an investment decision criterion for the design of the WDN. The proposed WDN optimization model is guided by its objective function, where new irrigation hydrants and respective pipelines are added to the optimal solution only if their discounted marginal benefit contribution is at least as great as the discounted marginal costs. This characteristic of optimization models was originally used by Bassoco et al. (1983) and Hazell and Norton (1986) to develop a procedure for project appraisal. The authors show a formal (mathematical) identity between the concept of IRR and the rate of return calculated by the optimization model. The following discussion is a modified version of these authors' original contribution, using a very simplified objective function.

If the condition for including one more irrigation hydrant h in the optimal solution of the proposed optimization model is such that discounted net-benefits are at least as large as the initial investment costs:

$$B_t(h) \cdot \sum_{t=1}^{T} \frac{1}{(1+r)^t} \geq I(h) \quad . \tag{11.1}$$

In the above expression, $B_t(h)$ are assumed uniform expected net-benefits in t of hydrant h. $I(h)$ represents the hydrant's initial investment costs and r is the cost of capital. Given the characteristics of the geometric series:

$$B_t(h) \cdot \frac{1 + \left(\frac{1}{1+r}\right)^{T-1}}{r} \geq I(h) \quad . \tag{11.2}$$

In the limit, the term $\left(\frac{1}{1+r}\right)^{T-1}$ converges to zero and in the optimal solution investments are chosen so that the recurrent net-benefit associated with the marginal amount of investment $I(h)$ are at least as great as the capital cost of this investment in t .

$$B_t(h) \geq rI(h) \tag{11.3}$$

Considering all hydrants H, the stream of cash-flows, and the total amount of investment gives:

$$\sum_{h=1}^{H} B_t(h) \cdot \sum_{t=1}^{T} \frac{1}{(1+r)^t} \geq \sum_{h=1}^{H} rI(h) \cdot \sum_{t=1}^{T} \frac{1}{(1+r)^t} \ , \tag{11.4}$$

which, after simplifying, equals:

$$B_p(h) = B_t^* \cdot \sum_{t=1}^{T} \frac{1}{(1+r)^t} \geq rI^* \cdot \sum_{t=1}^{T} \frac{1}{(1+r)^t} \ . \tag{11.5}$$

In the limit the present value of the benefits is greater than or equal to the present value of the investment

$$\lim_{T\to\infty} \quad B_p \geq rI^* \cdot \frac{1}{r} = I^* \ , \tag{11.6}$$

which is by definition the characteristic of the IRR, the discount rate at which an investment breaks even (Bassoco et al., 1983; Hazell and Norton, 1986).

Given this result, one can say that when setting alternative values of r in this kind of model, one is actually using alternative values of the IRR (Bassoco et al., 1983; Hazell and Norton, 1986). The process of working with the model will consist of setting the IRR at different levels and observing the level of investment, the number of irrigation hydrants in the optimal solution, their spatial distribution according to different elevations and distances to the pump, the different optimal diameters chosen and the optimal hydrant opening scheme in the different shifts. The variation of IRR and induced investments allows tracking of a schedule relating the NPV, IRR and optimal size (investment) of the water distribution network

The remainder of the chapter discusses in Section 11.2 how different relative magnitudes of the demanded IRR, escalating rates of real diesel prices (e_d) , real food prices (e_f), as well as different life-times of the investment (n), impact on the discounting factors accentuating more the benefits, the energy costs or more the initial investment costs in the NPV calculation. Section 11.3 introduces the model's scenarios for e_f and e_d, IRR and the life-time n of the investment, as well as for different realizations of the project's benefits (sensitivity analysis of benefit's assumed level).

11.2 The impact on discount factors of changes in the demanded IRR, escalation rates, and investment life-times

The objective of this section is to explore *a priori* the effects of changes in the relative magnitude of IRR, e and investment life-times n on the discounting process. Different parameterizations for optimization runs will be designed for validation of the model against these expected effects. Example calculations for $e = 0\%$, 1%, 3% and $e = 5\%$ were performed for IRR= 10%, 20%, 30%, and 40%, over life-times of the investment ranging from 5 to 30 years. The calculations were performed using Equations 11.7 and 11.8 (see also Chapter 7). The effects on the discounting factors (DF) can be seen in Figure 11-1 for the given escalation rates and 30 year project life-time. Further calculations with other project life-times are reported in Table 11.1

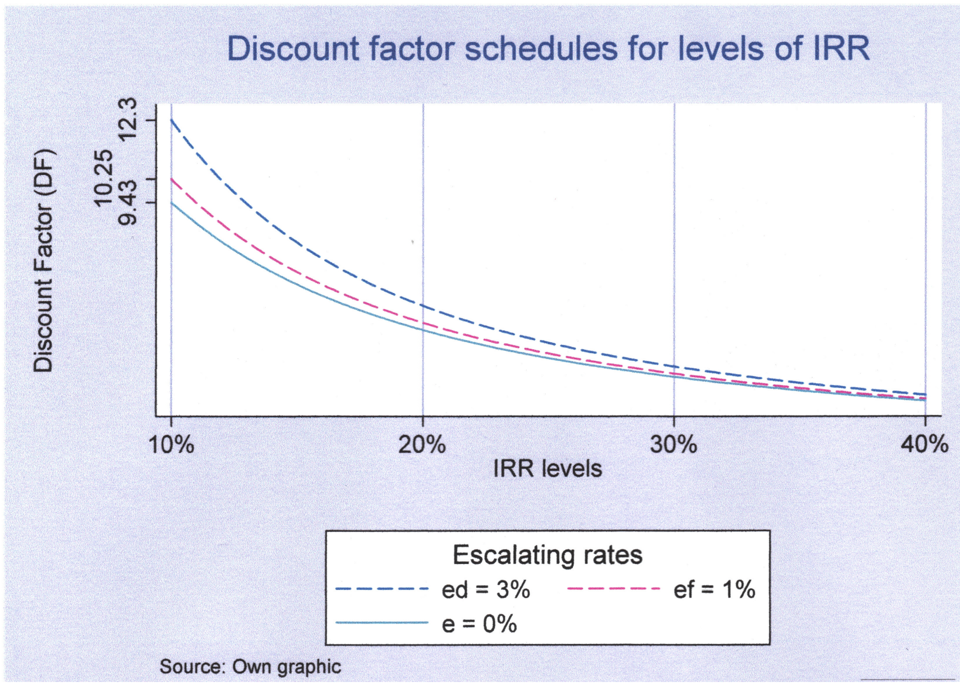
$$DF\left(e\right) = \frac{\left(1 + IRR\right)^n - \left(1 + e\right)^n}{\left(1 + IRR\right) - \left(1 + e\right)} \cdot \frac{1}{\left(1 + IRR\right)^n} \tag{11.7}$$

$$DF\left(e = 0\%\right) = \frac{\left(1 + IRR\right)^n - 1}{IRR \cdot \left(1 + IRR\right)^n} \tag{11.8}$$

As can be seen in Figure 11-1, cash-flows with higher escalating rates are in relative terms more weakly discounted for lower IRR and are in this case emphasized in the NPV calculation.

The difference converges to zero with increasing IRR levels, i.e. increasing the IRR will emphasize the initial investment costs, as well as the network's discounted O&M and depreciation costs ($e = 0\%$).

Figure 11-1: Discount factor schedules for different escalating rates



Expected trade-offs between initial investment costs (pipeline diameters) and diesel costs

If it is expected that the escalating rates of diesel prices are higher than those of food prices ($e_d > e_f$) diesel costs are emphasized for lower IRR levels and the model is expected to minimize diesel costs in two ways: **(1)** by producing designs that induce lower friction losses, i.e. selecting pipelines with **larger diameters; (2)** by determining a spatially more **concentrated pattern** of simultaneous irrigating hydrants (shift pattern).

With increasing demanded IRR the diesel costs are more strongly de-emphasized than the benefits while the difference between the two discount factors converges to zero. This fact will emphasize the weighting of the initial investment costs in the model's design. The model should determine a network design that minimizes the initial investment costs, i.e. that: **(1)** selects pipelines with **smaller diameters** and **(2)** determines a spatially more **distributed pattern** of simultaneous irrigating hydrants (shift pattern).

Table 11.1: Present value for different IRR, escalating price rates and life-times

IRR	Esc.Rates	5	10	15	20	25	30
	5%	4.151	7.440	10.046	12.112	13.749	15.046
10%	3%	4.003	6.884	8.958	10.450	11.525	12.299
	1%	3.860	6.379	8.023	9.096	9.796	10.253
	0%	3.791	6.145	7.606	8.514	9.077	9.427
	5%	3.247	4.913	5.767	6.205	6.430	6.545
20%	3%	3.142	4.606	5.288	5.605	5.753	5.822
	1%	3.040	4.324	4.867	5.096	5.192	5.233
	0%	2.991	4.192	4.675	4.870	4.948	4.979
	5%	2.625	3.527	3.838	3.944	3.981	3.993
30%	3%	2.547	3.343	3.591	3.669	3.693	3.700
	1%	2.472	3.172	3.370	3.426	3.442	3.447
	0%	2.436	3.092	3.268	3.316	3.329	3.332
	5%	2.179	2.696	2.819	2.848	2.855	2.857
40%	3%	2.120	2.577	2.676	2.697	2.701	2.702
	1%	2.063	2.466	2.545	2.560	2.563	2.564
	0%	2.035	2.414	2.484	2.497	2.499	2.500

With regarding to the network's size the model is expected to reduce the size of the network with increasing levels of IRR (dropping the most unfavourable hydrants). Two different patterns are expected to occur: **(1)** if the escalating diesel rates are higher than the escalating food price rates, reductions in the size of the network will occur for relatively lower levels of IRR; **(2)** if escalating rates of diesel prices are lower than those of the investment's benefits, reductions in the network's size are expected to occur for relatively higher levels of IRR.

11.3 Scenarios for model runs

The procedure for working with the model will be to set the IRR at different levels and observe the achieved NPV, the network's size (number of hydrants), shift patterns, pipeline dimensioning, as well as pressures and discharge balances. The exercise consists of parameterizing the model with IRR values ranging from 10% to 45% for assumed escalating rates of real diesel and food prices. Two main scenarios are derived for the optimization runs:

Scenario 1: *Diesel prices are assumed to have higher escalating rates than food prices during the life-time of the project.* The escalating rate for food prices e_f is set to 1%. The escalation rate for diesel prices is set to $e_d = 3\%$. The investment life-time (n) equals 30 years. This scenario imposes a conservative value for the diesel price escalation rate of $e_d = 3\%$, which is above the 30 year forecast $e_d = 1.6\%$ from Chapter 7, Section 7.3.1. The objective is to better explore the trade-offs between initial investment costs and energy costs.

Scenario 2: *Expected potential yields are not achieved and the expected benefits of each hydrant are lower than assumed. Diesel prices and food prices escalating rates are expected to remain at the same levels* ($e_f = 1\%$ and $e_d = 3\%$).

The goal of the proposed investment model is the **identification** of optimal WDN designs according to the proposed scenarios. The final design choice, and not the **identification**, may be made according to other complementary goals of the implementing agency, e.g. creation of employment.

Part IV

Output

Chapter 12

Results

12.1 Introduction

This chapter reports the results of the optimization model according to the scenarios discussed in Chapter 11. The results for Scenario 1 are presented in Section 12.2 in three different parts. Section 12.2.1 to 12.2.3 discuss the impacts of increasing IRR levels on the size of the network, on the spatial distribution of simultaneously irrigating hydrants, on the diameters of the network's pipelines, on the pressures at each section and hydrant, as well as the pipeline discharges. Detailed results are only presented for run IRR = 36% in the interest of space[1]. For this run, Section 12.2.4 presents an economic analysis including selected economic indicators of the investment, and also indicators relating to water and energy use performance. For a better assessment of the plausibility of the optimized designs, results on water and energy use indicators are compared with a benchmark irrigation scheme, also working under a rotational water delivery management.

The results for Scenario II are presented in Section 12.3, where a sensitivity analysis shows how changes in the expected potential net return of farming activities impact on the NPV and justifiable investments (number of hydrants in the optimal solution).

[1]Results for another selected run (IRR=32%) can be seen in Appendix A. Results for any other run can be delivered on request.

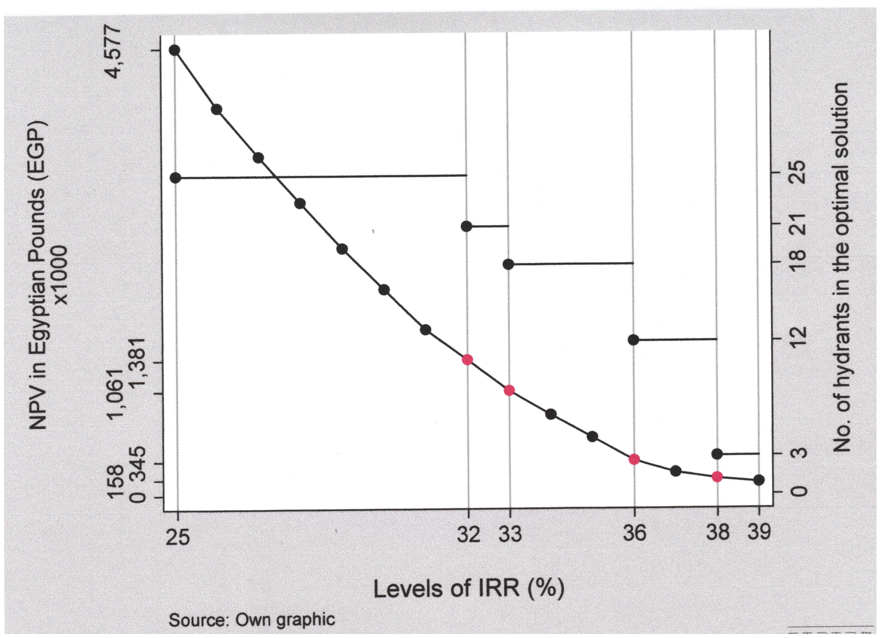
12.2 Optimization results for Scenario I

Scenario 1: *Diesel prices are assumed to have higher escalating rates than food prices during the life-time of the project.* The escalating rate for food prices e_f is set to 1% and for diesel prices is set to $e_d = 3\%$. The investment life-time (n) equals 30 years.

12.2.1 The network's size

The results show that the size of the network will be quite sensitive to IRR levels higher than 32%. Figure 12-1 gives the development of the investment's NPV and size of the optimal network in a 'nut-shell'. The red dots indicate a reduction in size of the network with the drop of several hydrants for the given IRR level.[2]

Figure 12-1: Net-Present-Value schedule and No. of hydrants for different IRR levels



[2]Figure 12-1 presents the NPV-schedule only for $IRR \geq 25\%$. Lower IRR values produce of course higher NPV levels.

For IRR$<$ 32% the network departing size remains unchanged with 25 hydrants (see Figure 12-2). For IRR = 32% the network size is reduced by four hydrants (number 19, 26, 27, and 30), which are dropped from the optimal solution together with their connecting pipelines (see Figure 12-3). The dropped hydrants were the most "unfavourable" ones, where elevations or distances to the pump system are higher demanding more energy due to higher head losses. Further increasing the IRR to 33% induces again a reduction in the justifiable amount of investment dropping three more hydrants (number 6, 11, and 25 in Figure 12-4). Again the model shows its accuracy by choosing the hydraulically most unfavourable nodes, i.e. where either elevations or distances to the pumping system are more unfavourable. With IRR = 36% branches VI, and VII, are totally dropped from the solution. The other branches I to V remain but the IRR level does not justify hydrants number 10, 18 and 22, and these are dropped (Figure 12-5). For IRR = 38% only three hydrants remain in the optimal solution (Figure 12-6). Further increasing the IRR would not justify any more investments and no hydrants are found in the optimal solution.

12.2.2 The spatial distribution of simultaneously irrigating hydrants: the shift patterns

The proposed model simultaneously optimizes the network's size, pipeline diameters, demanded pumping system capacity, and the optimal spatial distribution of irrigating hydrants in each shift. To this author's knowledge, no other irrigation WDN optimization model has addressed these problems simultaneously. The best shift pattern for operating hydrants is the one that maximizes the NPV objective function of the proposed model, whilst balancing the trade-offs between initial investment costs in the network's pipelines (diameters), pumping system and recurrent costs (e.g. diesel). As discussed in Chapter 6, the proposed WDN operates on three shifts, and has a potential size of 25 hydrants, i.e. there are $8.5 \cdot 10^{11}$ possible shift patterns (see Chapter 6). The problem in this dissertation is even more complex because the size of the network is endogenous, i.e. the total number of hydrants is a variable that changes according to the justifiable amount of investment.

In **Scenario I** the escalating rates of diesel prices are higher than the assumed escalating rates of hydrants benefits ($e_d = 3\%$ and $e_f = 1\%$) . As discussed in Section 11.2, this means that *diesel costs* are **emphasized** more for *lower* IRR *levels* and the model is expected to minimize diesel costs in two ways: **(1)** by producing designs that induce lower friction losses, i.e. selecting the pipelines with **larger diameters**, and **(2)** by determining a spatially more **concentrated shift pattern**. This is exactly what can be seen in the model runs for IRR $< 32\%$ (all these runs show the same shift patterns as in IRR $= 25\%$ in Figure 12-2). Furthermore, given this pattern we have higher discharges in the branches and the model chooses a network design with diameters larger than those of higher IRR runs (see Figure 12.2). Higher diameters imply lower friction losses and reduce diesel costs, but are on the other hand more expensive (trade-off between diesel costs and initial investment costs). With increasing IRR the diesel costs are more strongly **de-emphasized** than the hydrant's benefits and the difference between the two discount factors converges to zero (see Figure 11-1). This fact will emphasize the weighting of the *initial investment costs* in the model's NPV calculation. The model should determine network designs that *minimize the initial investment costs*, i.e. that **(1)** select pipelines with **smaller diameters** and **(2)** determine a spatially more **distributed shift pattern**. This expectation is confirmed in the designs with increasing IRR (compare Figures 12-3 and 12-4 with Figure 12-2).

These interactions between initial investments, diesel costs, and IRR levels would be even more clear if the size of the network would be fixed whilst IRR levels are increased. Because this model is in the position of determining the optimal size of the network according to the IRR level, the model shows one more interesting characteristic: the model creates for IRR $= 33\%$ two different pressure zones balancing head losses according to the distance to the system's pump. Branches nearer to the pump present a more distributed shift pattern, which helps reduce surplus pressure at the respective hydrants. More unfavorable branches present a more concentrated shift pattern, accommodating higher head losses due to longer paths (see Figure 12-4).

Figure 12-2: Optimal network size and hydrant schedule for IRR = 25

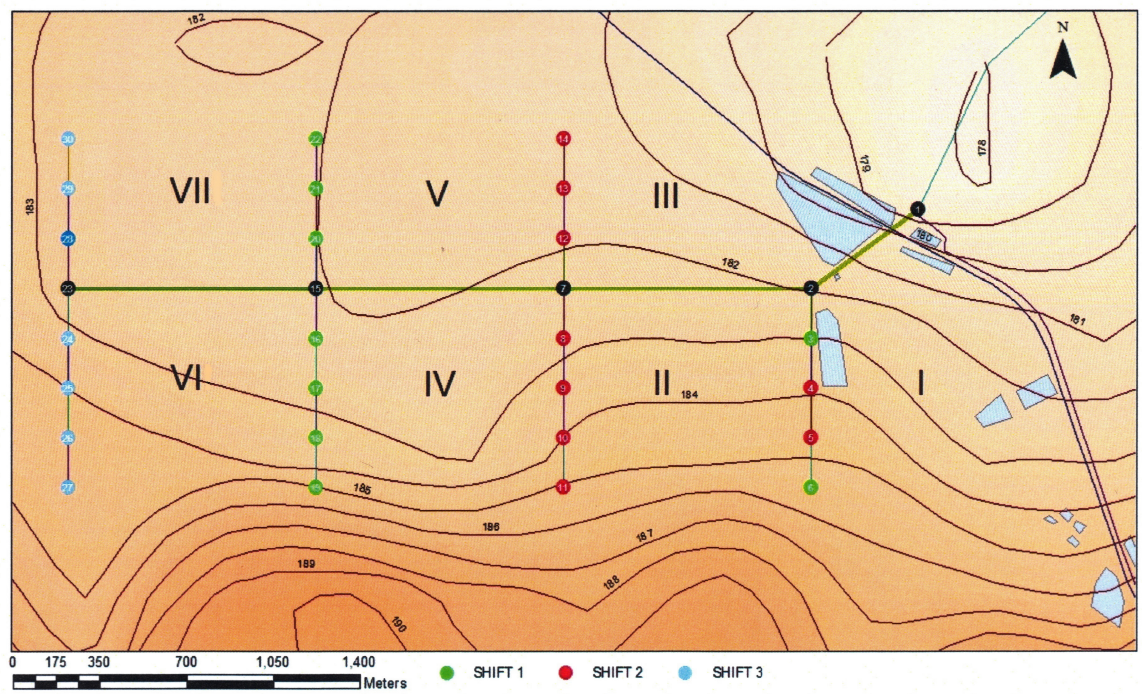


Figure 12-3: Optimal network size and hydrant schedule for IRR = 32

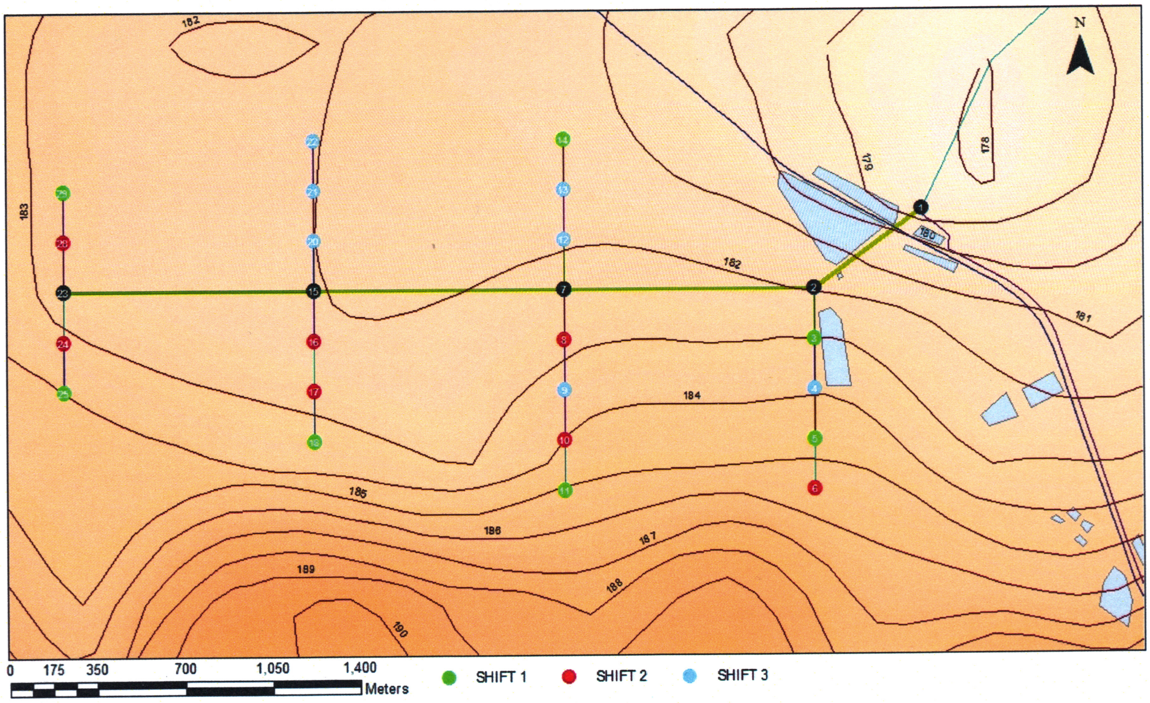


Figure 12-4: Optimal network size and hydrant schedule for IRR = 33

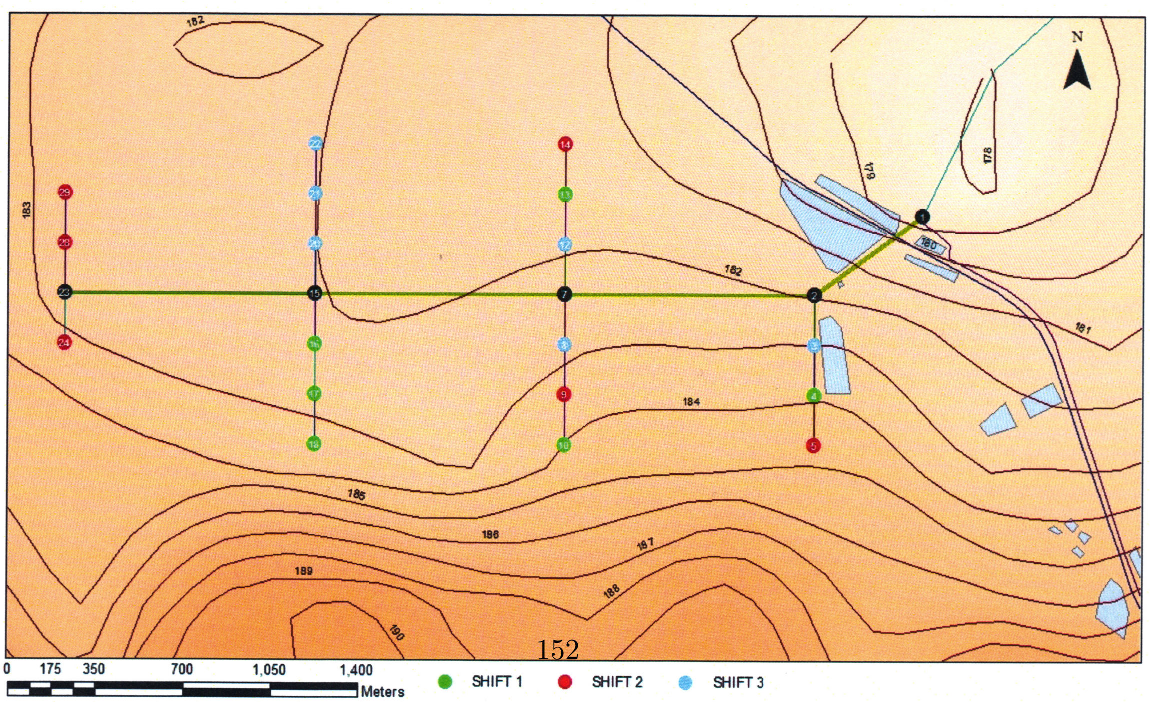


Figure 12-5: Optimal network size and hydrant schedule for IRR = 36

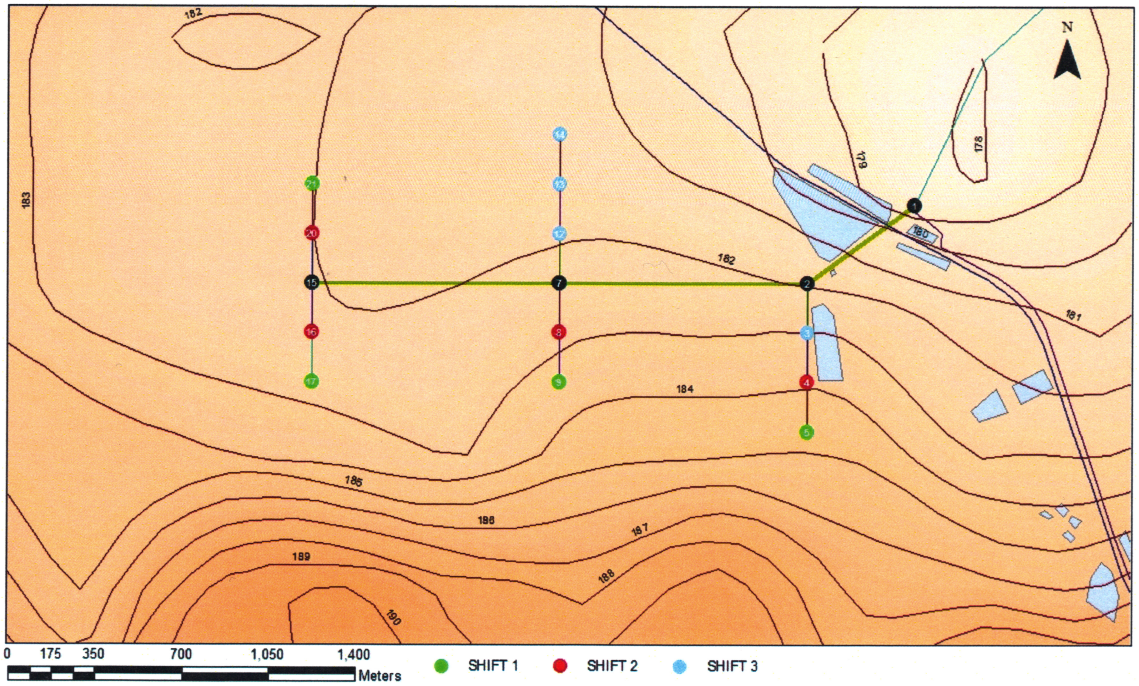
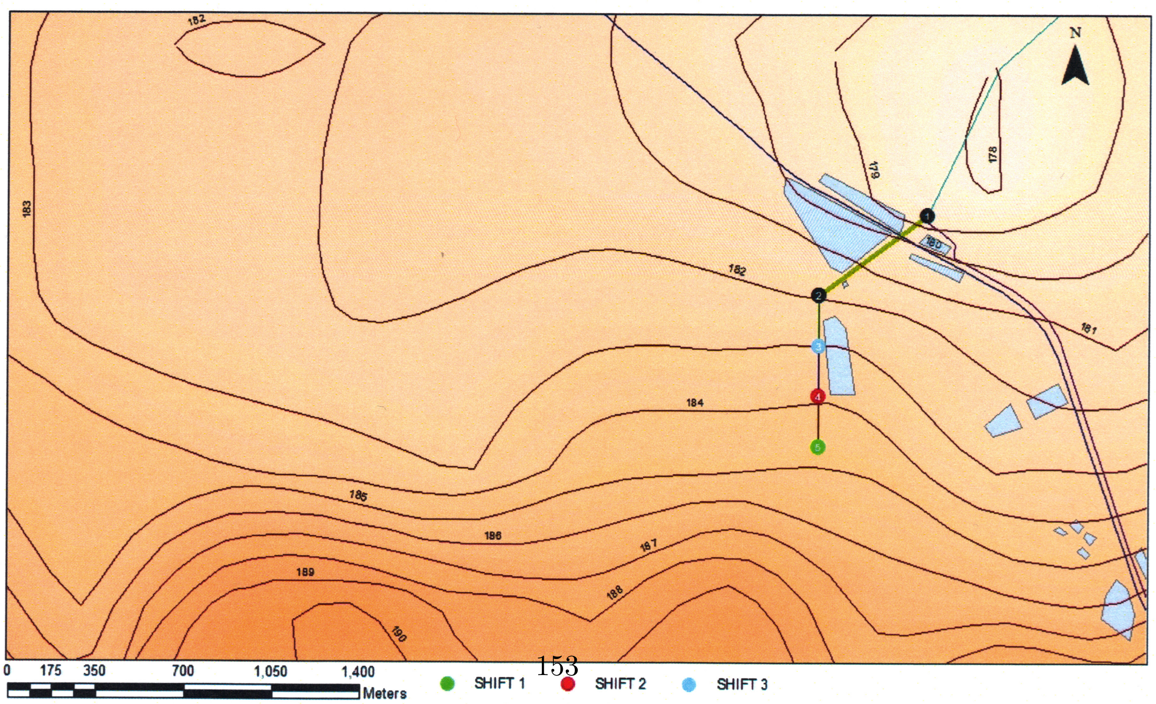


Figure 12-6: Optimal network size and hydrant schedule for IRR = 38



12.2.3 Diameters, pressures and discharges of the network design justified with IRR = 36%

The model chooses the most suitable diameters according to the discharges in the pipelines (law of mass conservation) corresponding to the selected shift patterns (Table 12.1). The model shows a coherent selection of pipes with diameters graduating in size, and becoming smaller the further downstream they are in the branch. The model performs this choice in a 'natural' way, i.e. in the optimization process. There are no constraints imposing this condition. Recent meta-heuristic methods dimensioning branched irrigation networks show difficulties in producing such patterns and have to impose constraints *a priori* to assure that downstream pipelines have smaller diameters than upstream ones (Farmani et al., 2007).

Table 12.1: Pipeline's diameters: optimization run r=36, ef=1, ed=3 and n=30

29-30	0 cm	21-22	0 cm	13-14	44 cm		
28-29	0 cm	20-21	39 cm	12-13	62 cm		
23-28	0 cm	15-20	44 cm	7-12	80 cm		
15-23	0 cm	7-15	62 cm	2-7	80 cm	1-2	80 cm
23-24	0 cm	15-16	44 cm	7-8	39 cm	2-3	44 cm
24-25	0 cm	16-17	44 cm	8-9	39 cm	3-4	44 cm
25-26	0 cm	17-18	0 cm	9-10	0 cm	4-5	44 cm
26-27	0 cm	18-19	0 cm	10-11	0 cm	5-6	0 cm

The expected trade-offs between initial investment (diameters) and the diesel costs are also evident in the diameter dimensioning through the different IRR runs. For increasing levels of IRR the discount factor for diesel costs are de-emphasized more than the discount factor for the network's benefits, which results in a stronger weighting for the initial investment costs in the model's NPV calculation. Given this, the model produces network designs showing decreasing average diameters of the pipelines, as can be seen in Table 12.2.

The run IRR = 33% tends to contradict the above statement at first glance, because the average diameter actually slightly increases from IRR = 32% to IRR = 33%. This occurs only because the size of the network is endogenous[3]. The model decides in this case to reduce the size of the network and creates two pressure zones for balancing head losses with higher diameter averages in branches V-VII and lower in branches I-IV. Further increases in the IRR will continue to reduce the average diameters of the pipeline network as expected.

Table 12.2: Average pipeline diameters for the selected runs

IRR (%)	25	32	33	36	38
Average diameters (cm)	66	51	55	53	44

The optimized pressures across the nodes of the network can be seen in Table 12.3, also showing a very plausible picture. The model determines the minimum requested pressure of 40 m for the most unfavorable hydrant in each shift. There are no relevant overpressures at the hydrants; this fact is of great importance showing the energy efficiency of the optimal solution. The model is absolutely flexible, allowing different demanded heads and flow regimes from the pumping system in each shift.

The model also correctly optimizes the discharge balances (conservation of mass) across the nodes of the network (Table 12.4). In the case of run IRR = 36% we have an optimal size of 12 hydrants, in this case a shift pattern of four hydrants irrigating per shift was implemented, the pumping system works with a discharge of 720 $\mathrm{L \cdot s^{-1}}$ in all shifts, an even number of hydrants irrigating simultaneously.

[3]Forcing the size of the network to be constant for increasing levels of IRR would reduce the pipeline average diameters in the 33% case.

Table 12.3: Nodes' pressures for run: r=36, ef=1, ed=3, and n=30

	30	0 m	22	0 m	14	0 m		
	29	0 m	21	40.2 m	13	0 m		
	28	0 m	20	40.9 m	12	0 m	1	48 m
SHIFT 1	23	0 m	15	41.4 m	7	42.9 m	2	44.2 m
	24	0 m	16	40.9 m	8	41.5 m	3	42.6 m
	25	0 m	17	40 m	9	40.2 m	4	41.3 m
	26	0 m	18	0 m	10	0 m	5	40.1 m
	27	0 m	19	0 m	11	0 m	6	0 m
	30	0 m	22	0 m	14	0 m		
	29	0 m	21	0 m	13	0 m		
	28	0 m	20	40 m	12	0 m	1	47.1 m
SHIFT 2	23	0 m	15	40.4 m	7	42 m	2	43.2 m
	24	0 m	16	40 m	8	40.6 m	3	41.7 m
	25	0 m	17	0 m	9	0 m	4	40.3 m
	26	0 m	18	0 m	10	0 m	5	0 m
	27	0 m	19	0 m	11	0 m	6	0 m
	30	0 m	22	0 m	14	40 m		
	29	0 m	21	0 m	13	40.4 m		
	28	0 m	20	0 m	12	40.7 m	1	45.8 m
SHIFT 3	23	0 m	15	0 m	7	40.7 m	2	42 m
	24	0 m	16	0 m	8	0 m	3	40.4 m
	25	0 m	17	0 m	9	0 m	4	0 m
	26	0 m	18	0 m	10	0 m	5	0 m
	27	0 m	19	0 m	11	0 m	6	0 m

Table 12.4: Pipelines discharges in L/s for the run r=36 ef=1 ed=3 and n=30

SHIFT 1							
29-30	0 L/s	21-22	0 L/s	13-14	0 L/s		
28-29	0 L/s	20-21	180 L/s	12-13	0 L/s		
23-28	0 L/s	15-20	180 L/s	7-12	0 L/s		
15-23	0 L/s	7-15	360 L/s	2-7	540 L/s	1-2	720 L/s
23-24	0 L/s	15-16	180 L/s	7-8	180 L/s	2-3	180 L/s
24-25	0 L/s	16-17	180 L/s	8-9	180 L/s	3-4	180 L/s
25-26	0 L/s	17-18	0 L/s	9-10	0 L/s	4-5	180 L/s
26-27	0 L/s	18-19	0 L/s	10-11	0 L/s	5-6	0 L/s
SHIFT 2							
29-30	0 L/s	21-22	0 L/s	13-14	0 L/s		
28-29	0 L/s	20-21	0 L/s	12-13	0 L/s		
23-28	0 L/s	15-20	180 L/s	7-12	0 L/s		
15-23	0 L/s	7-15	360 L/s	2-7	540 L/s	1-2	720 L/s
23-24	0 L/s	15-16	180 L/s	7-8	180 L/s	2-3	180 L/s
24-25	0 L/s	16-17	0 L/s	8-9	0 L/s	3-4	180 L/s
25-26	0 L/s	17-18	0 L/s	9-10	0 L/s	4-5	0 L/s
26-27	0 L/s	18-19	0 L/s	10-11	0 L/s	5-6	0 L/s
SHIFT 3							
29-30	0 L/s	21-22	0 L/s	13-14	180 L/s		
28-29	0 L/s	20-21	0 L/s	12-13	360 L/s		
23-28	0 L/s	15-20	0 L/s	7-12	540 L/s		
15-23	0 L/s	7-15	0 L/s	2-7	540 L/s	1-2	720 L/s
23-24	0 L/s	15-16	0 L/s	7-8	0 L/s	2-3	180 L/s
24-25	0 L/s	16-17	0 L/s	8-9	0 L/s	3-4	0 L/s
25-26	0 L/s	17-18	0 L/s	9-10	0 L/s	4-5	0 L/s
26-27	0 L/s	18-19	0 L/s	10-11	0 L/s	5-6	0 L/s

12.2.4 Economic analysis of the investment justified for IRR = 36%

For IRR = 36%[4] the total justifiable initial investment amounts to 691,568 EUR, and 12 hydrants are implemented (Figure 12-5). This corresponds to a total agricultural area of 240 ha, i.e. 120 farms and a total of 46,560 man-days of work per year for the whole settlement[5]. The network design justified by the IRR = 36% level shows an NPV of 180 $EUR \cdot ha^{-1}$. This is calculated as the difference between the discounted benefits 7,030 $EUR \cdot ha^{-1}$ and discounted total costs of 6,850 $EUR \cdot ha^{-1}$ (the sum of discounted costs of diesel, O&M, depreciation and initial investments). The discounted costs of diesel are by far the most relevant cost position with 939,905 EUR for the 30 year discount period.

Table 12.5: Investment results for optimization run r=36, ef=1, ed=3, and n=30

	EGP	*EUR*
Net present value	345,185	43,148
discounted node's net-benefits	13,496,835	1,687,104
discounted energy costs	7,519,237	939,905
discounted maintenance costs	21,827	2,728
discounted depreciation costs	78,043	9,755
Initial investment costs		
pipeline network	600,030	75,004
pump system	242,913	30,364
farm's irrigation systems	4,689,600	586,200
Total initial investment costs	5,532,543	691,568
Specific costs per year		
maintenance costs	7,858	982
depreciation	28,098	3,512

[4] Results for any other run can be delivered on demand. Given the space limitations, only one run could be presented with detail. There was no particular reason for choosing run $IRR = 36\%$ in detriment of the other runs.

[5] The basis for calculation is the 388 man-days of work per year and per farm discussed in the farm budget of Chapter 7.

Water and energy use indicators (benchmarking)

The model should produce designs giving performances comparable with other irrigation schemes also operating on a rotational water delivery management system. Because no data could be found in Egypt for a similar irrigation scheme on a rotational management, the study of Moreno et al. (2010) for a scheme in the Spanish region of Castilla-La Mancha (CLM) was chosen for benchmarking. In reality, the two case-studies are not directly comparable given the different countries, soils, climatic conditions, etc. The Kalabsha (KLB) case-study is situated in Aswan under very unfavorable arid desert conditions, with much higher evapotranspiration rates than the Spanish case. Furthermore, the benchmark scheme is real and actively managed according to the climatic conditions prevailing through the different months of the year (e.g. control of irrigation interval, see Chapter 6). The proposed model calculates irrigation intervals based on average crop season values and historical weather data from a regional weather station. This can introduce bias in the irrigation schedule calculation resulting in some overestimation of the annual operating hours and diesel costs. These facts should be taken into account when making comparisons, and it should be clear in this section that the purpose of this comparison is solely to assess the plausibility of the results proposed by the optimization model of this dissertation, allowing a better judgment of the optimized WDN design.

For the assessment, selected resource use indicators based on Moreno et al. (2010) were utilized to evaluate water and energy consumption, as well as the energy costs of each of the case-studies. The results on the selected indicators for the CLM scheme are presented together with the KLB model run results for IRR = 36% in Table 12.6.

The annual cropping area (S_r) proposed by the model run for IRR = 36% is 480 ha (two seasons). The CLM irrigation scheme has a total area of ca. 640 ha.

The annual volume of irrigation water per unit of area (V_tS_r) is ca. 1.6 times higher in the KLB than in the CLM case. This high number for the KLB case is not surprising, given the hotter climatic conditions in Kalabsha. It is also justified by the conservative choice in the optimization model of crops with high water demands for calculating the evapotranspiration and irrigation intervals in Chapter 6 (the use of high average crop coefficients). In the CLM case the seasonal crops were most probably not as water-demanding as those chosen for modelling the KLB case[6].

Average demanded power (AP) is ca. 825.5 kW for KLB against 365.4 kW for the CLM case. The high level for KLB is justified by the low power efficiency of the off-grid system (see further explanations below and in Chapter 5).

The annual energy consumed per irrigation supply (CEV_t), is 0.32 $\text{kWh} \cdot \text{m}^{-3}$ for KLB and 0.266 $\text{kWh} \cdot \text{m}^{-3}$ for CLM, a difference of 0.054 $\text{kWh} \cdot \text{m}^{-3}$.

The annual energy consumed per irrigated area (CES_r) for KLB was 3,823 $\text{kWh} \cdot \text{ha}^{-1}$, and for CLM 2,197 $\text{kWh} \cdot \text{ha}^{-1}$. The result shows a similar proportionality to that for $V_t S_r$, and again the model results are very plausible given the large climatic difference between the two regions.

The diesel costs per irrigated area DCS_r, show a value of 646 $\text{EUR} \cdot \text{ha}^{-1}$ for KLB and 220 $\text{EUR} \cdot \text{ha}^{-1}$ for CLM. It should be mentioned here that in the KLB case we are referring to diesel, while in the CLM case we have electricity costs. The KLB irrigation system is implemented in an off-grid settlement. This implies the use of diesel generators to produce electricity.

Diesel costs per irrigation supply (DCV_t) show 0.054 $\text{EUR} \cdot \text{m}^{-3}$ for KLB. The electricity costs for the CLM case were 0.0296 $\text{EUR} \cdot \text{m}^{-3}$.

All the calculated indicators for the KLB dimensioned WDN system are ca. 1.5 to 1.8 times higher than in the CLM system, but nevertheless very plausible. One has to consider the different climatic conditions and crop choices in the two cases and the fact that the KLB system belongs to an off-grid settlement, which conditions strongly the power efficiency. In the CLM case, the power system consists of electrical motors connected to the region's grid, coupled with respective centrifugal pumps. In the case of the KLB irrigation

[6] There is no information about the crop coefficients used in Moreno et al. (2010).

scheme, we are dealing with diesel generators supplying electricity to the electric motors of the centrifugal pumps. This means that in the CLM scheme we have two efficiencies to deal with, whereas in the KLB system we have three different components in the power production (diesel generator, electric motor and centrifugal pump). We are dealing here with three different efficiencies that need to be multiplied with each other, which will surely result in an overall lower efficiency than for the CLM case[7] (see also Chapter 5 for a detailed discussion on system's power efficiency).

[7] There is no information about the power system's efficiency in the Moreno et al. (2010) study.

Table 12.6: Water and energy use indicators for run r=36, ef=1, ed=3, and n=30

	IRR= 36%	Moreno et al. (2010)	Units
Irrigated area (S_r)	480*	640	ha
Total annual volume of irrigation water supply (V_t)	5,762,534	5,284,266	m^3
Total annual volume of irrigation water supply per irrigated area ($V_tS_r = V_t/S_r$)	12,005	7,429	$m^3 \cdot ha^{-1}$
Average demanded power (DP)	825.483	365.4**	kW
Average power per irrigated area ($DPS_r = DP/S_r$)	1.72	0.571***	$kW \cdot ha^{-1}$
Total hours of operation per year ($Thrs$)	2,223.2	na	h
Consumed energy ($CE = DP \cdot Thrs$)	1,835,214	1,406,260	kWh
Consumed energy per irrigation supply ($CEV_t = CE/V_t$)	0.32	0.266	$kWh \cdot m^{-3}$
Consumed energy per irrigated area ($CES_r = CE/S_r$)	3,823	2,197	$kWh \cdot ha^{-1}$
Equivalent consumed diesel (ECD)	422,099	na	L
Diesel costs per irrigation water supply ($DCV_t = ECD \cdot p_d/V_t$)	0.0538	0.0296	$EUR \cdot m^{-3}$
Diesel costs per irrigated area ($DCS_r = ECD \cdot p_d/S_r$)	646	220	$EUR \cdot ha^{-1}$
Benefits per irrigated area ($BS_r = B/S_r$)	$1,230^H$	na	$EUR \cdot ha^{-1}$
Benefits per irrigation water supply ($BV_t = B/V_t$)	0.102	na	$EUR \cdot m^{-3}$

* Two crop seasons
**Average measured absorbed power
***Average measured absorbed power irrigated area
$p_d = 0.735$ EUR $\cdot$ L^{-1}
B = 49,214 EUR multiplied by No. Hydrants in the optimal solution
H = 12 hydrants in the optimal solution

Analysis of cost recovery potentials

Cost recovery in public irrigation projects is not an easy matter. Farmers are the direct beneficiaries of irrigation projects, but the question as to whether farmers should be charged for the full cost of the project cannot have a straightforward answer. Many authors discuss the fact that the private/public nature of such irrigation projects produces much broader long term economic returns than those accounted for in the project's CBA (positive externalities). There is a strong social nature of irrigation economic returns that may justify government subsidization. Many other indirect beneficiaries exist that profit from the development of irrigation, e.g. traders, processors, transporters and even consumers in the region (Small, 1996; Kulshreshtha, 2002). Should these also be charged for cost recovery?

Many authors argue that indirect beneficiaries should be happy to contribute to cost recovery in irrigation through the tax system (Small, 1996; Bhattarai et al., 2003). Molle and Berkoff (2007) discuss this justification for charging indirect beneficiaries stating that it is not as clear as it may seem at the first glance, among other arguments the authors remind that the project's multiple benefits should be seen in the light of incremental impacts relative to alternative irrigation projects. Reviewing studies on cost recovery of investments in irrigation schemes, in developing as well as developed countries, shows evidence that only a fraction pumping and O&M costs can normally be recovered, i.e. capital costs are not recovered at all in the large majority of public financed irrigation schemes (Molle and Berkoff, 2007).

Cost recovery potential for pumping costs are addressed here for the selected $IRR = 36\%$ design. The total farm area in the KLB settlement is 2 ha. From Table 12.6 the total annual water used is $12,005 \cdot 2 = 24,010$ m^3. Charging farmers for the costs of pumping water could be done on the basis of the calculated diesel costs per m^3, i.e. $24,010 \cdot 0.0538 = 1,292$ EUR or ca. $10,334$ EGP per year[8].

[8] The prevalent exchange rate in 2010 was on average 8 EGP per EUR.

The estimated average diesel costs per m^3, decrease with increasing IRR levels and smaller WDN sizes for the runs in Scenario I, as can be seen in Table 12.7.

Table 12.7: Diesel costs for different IRR levels and WDN designs

IRR (%)	25	32	33	36	38
Diesel costs ($EUR \cdot m^{-3}$)	0.0562	0.0559	0.0545	0.0538	0.0519

Annual maintenance costs were assumed in Chapter 7 as 0.5% of initial investment costs in the pipeline network and 2% of initial investment costs in the pumping system. The sum of the initial investment costs for these components is 5,532,543 EGP (see Table 12.5), which implies total maintenance costs of ca. 31,306 EGP per year[9]. Given 12 hydrants and 120 farms in the optimal solution, each farm should be charged ca. 261 EGP maintenance costs per year. The whole diesel and non-energy O&M costs per farm and per year would be 10,595 EGP (10,334 EGP + 261 EGP).

The potential annual income per farm in the KLB settlement was estimated in Chapter 7 as 39,371 EGP. Charging local farmers for pumping and non-energy O&M costs would give a net income of $39,371 - 10,595 = 28,776$ EGP per *year*, equivalent to 3,597 EUR per *year*, approximatly 300 EUR per *month*.

This net-return represents a quite modest return to a farmer's land, family work and management, indicating that pumping and non-energy O&M cost recovery would not be possible for the WDN and desert land reclamation project in this region. This is the most pertinent result if the true diesel costs for the implementing agency are considered with the help of shadow prices in the CBA of the project.

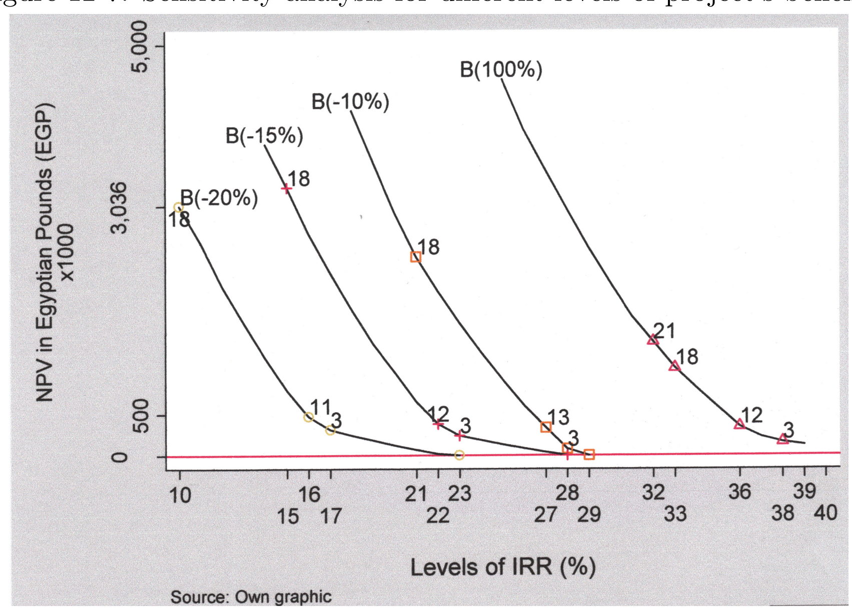
[9] The calculation basis for the maintenance costs at the farm level also includes the farms irrigation systems initial investment costs.

12.3 Optimization results for Scenario II

Scenario II: *Expected potential yields are not achieved and the expected benefits of each hydrant are lower than assumed. Diesel prices and food price escalating rates are expected to remain at the same levels* ($e_f = 1\%$ and $e_d = 3\%$).

This scenario is embedded in a **sensitivity -analysis** for the investment NPV and design of the network (level of investment, i.e. number of hydrants). The sensitivity analysis should give a feel for how the results might be affected by changes in the expected project's benefits. In order to analyse these impacts several model runs were performed for different assumptions of the project's benefits. In Figure 12-7 we compare the NPV and number of hydrants in the optimal solution for the different levels of achievable benefits (number of hydrants are indicated near the scatter points in the figure). The potential level of benefits B(100%) is compared with reductions of 10% B(-10%), 15% B(-15%) and 20% B(-20%).

Figure 12-7: Sensitivity analysis for different levels of project's benefits



For each level of assumed benefits, Figure 12-7 shows the level of IRR for which the design is first reduced from its maximal number of 25 hydrants[10]. For B(100%) there is a first reduction to 21 hydrants for $IRR = 32\%$, 18 hydrants are justifiable for $IRR = 33\%$. By reducing the level of benefits by 10%, B(-10%), the level of investment is reduced from 25 to 18 hydrants for a much lower IRR level (21%). For a 20% reduction, B(-20%), the reduction to 18 hydrants occurs already for $IRR = 10\%$. It can be seen that the level of investment and design of the network is quite sensitive to the level of benefits. There is a near 'elastic relationship' between the percentage change in benefits and the percentage change in the critical level of IRR for which the first reductions in the size of the network occur.

The sensitivity results clearly show that investments in irrigation systems should not be done on the basis of minimizing the network costs alone. The size of the network and different designs are very sensitive to the benefits and levels of IRR. Including the potential generated benefits of the investment is crucial for understanding the economic feasibility of the investment, for understanding the best design that should be implemented given the border conditions and demanded project returns. Furthermore, the precise estimation of the potential achievable yields emerges here as a very important project appraisal task. Considerable savings can be made if the project data can be refined, reducing insecurity[11].

In the following, detailed results for the B(-20%) case are given for a more detailed illustration[12].

[10] In Figure 12-7, all the NPV-IRR schedules are presented only for IRR values higher than a certain level. The purpose was to avoid too many optimization runs. For each schedule the NPV would of course increase for lower IRR values.

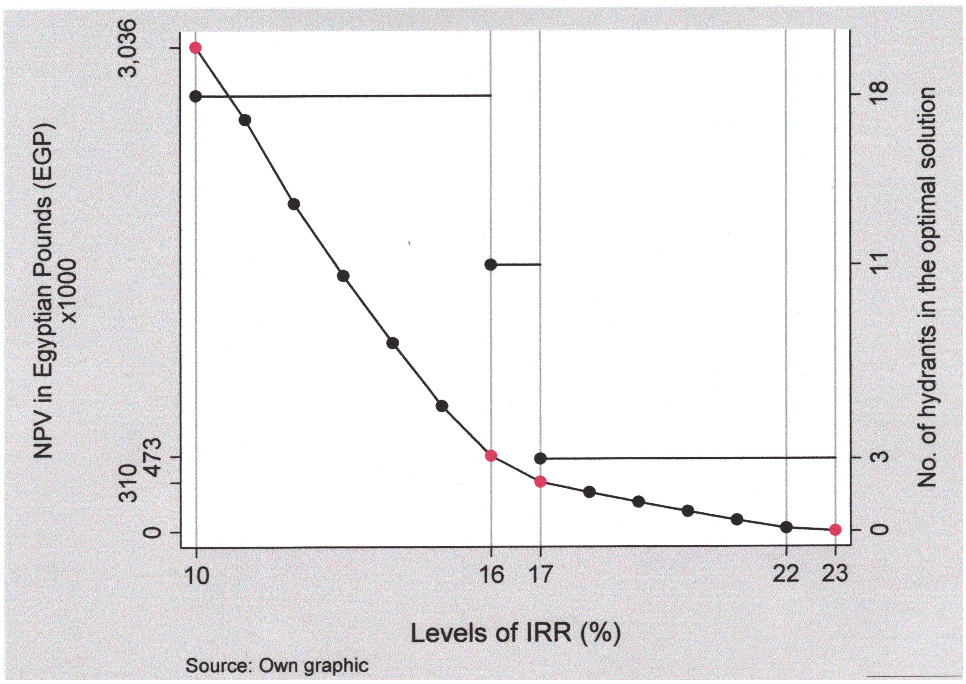
[11] Insecurities on the estimation of project implementation costs also exist. Nevertheless, these are not considered to have as much importance as the ones related to the agricultural activity.

[12] Results on any other run can be delivered on demand.

Detailed results for B(-20%) runs

For this case the model reduces the size of the network for much lower levels of IRR (see Figure 12-9). For $IRR = 10\%$ only 18 hydrants remain in the optimal solution, further reductions in investment and number of hydrants occurs for $IRR = 16\%$, and $IRR = 17\%$. For a level of $IRR = 23\%$ no investment is justifiable, there are no hydrants in the optimal solution.

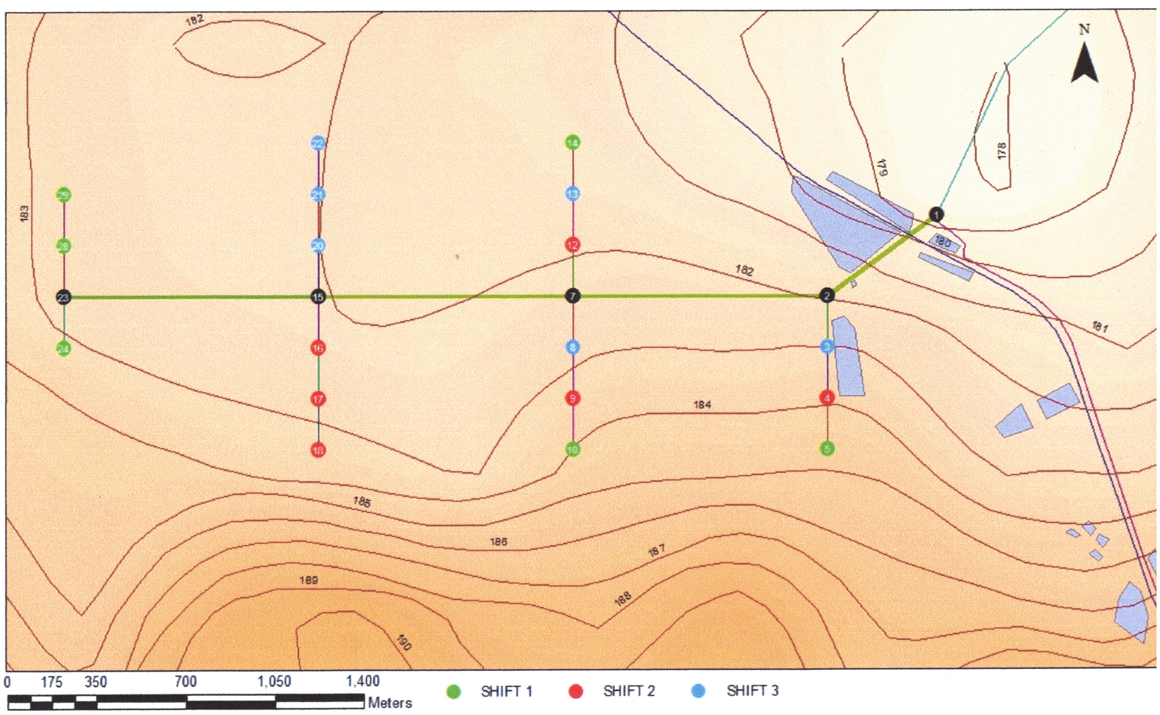
Figure 12-8: NPV schedule and No. of hydrants for different IRR levels



For $IRR = 10\%$ hydrants No. 30, 27, 26, 25, 19, 11 and 6 are not in the optimal solution. Regarding the shift patterns, the design for $IRR = 10\%$ represents a transition phase as in Scenario I $IRR = 33\%$, with two different shift pattern zones (Figure 12-9). The branches IV-VII present an concentrated shift pattern and the branches I-III a more distributed one. For $IRR = 16\%$ the branches VI and VII are not in the optimal solution.

From the remaining branches hydrants No. 22, 18, 10 and hydrant No. 5 are also dropped. For IRR = 17% only branch I remains in the solution with hydrants No. 5, 4 and 3. Further increases of the IRR level lead to zero justifiable investments, i.e. no hydrants in the optimal solution.

Figure 12-9: Optimal network size and hydrant schedule for IRR = 10



Further detailed results for Scenario II regarding economic parameters, water and energy use, as well as results for optimal pipeline diameters and networks discharges can be seen in Appendix B for the run $IRR = 16\%$.

Figure 12-10: Optimal network size and hydrant schedule for IRR = 16

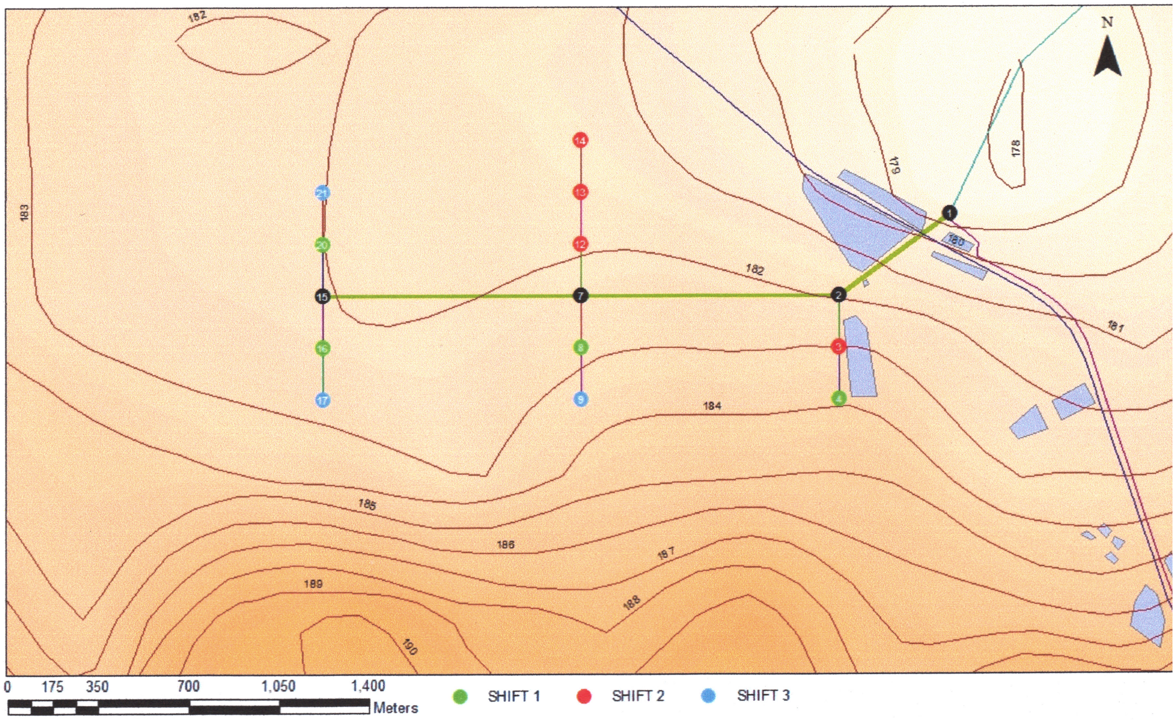
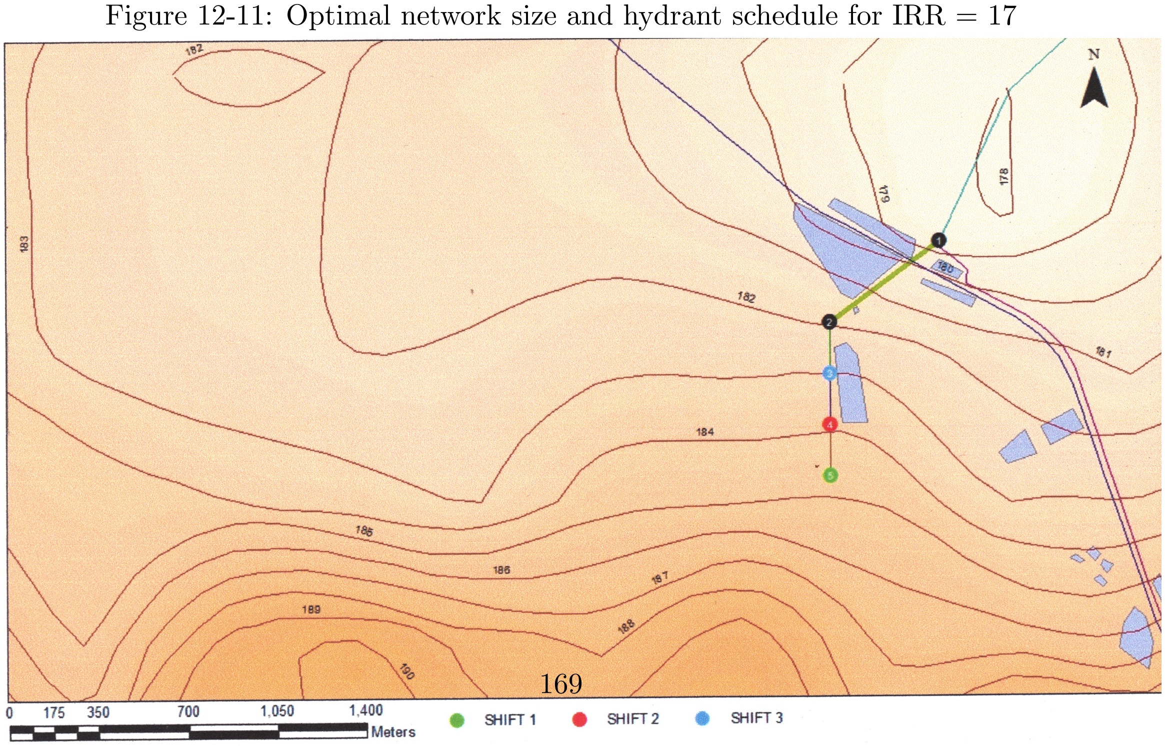


Figure 12-11: Optimal network size and hydrant schedule for IRR = 17



Chapter 13

Conclusions

The main goal of this dissertation was to close the methodological gap between WDN designs based only on minimization of the networks' costs and the necessity for more inclusive approaches based on cost-benefit analysis (CBA). A CBA optimization framework was developed balancing the life-cycle benefits, life-cycle costs and initial investments of the WDN project. The optimization model developed delivers the optimal network size, pipeline diameters, pump system capacity (discharges, heads and power demanded per shift), as well as the optimal spatial distribution of simultaneous irrigating hydrants per shift. The results and other important issues are summarized in the following points:

1. A comprehensive literature review on dimensioning WDN was performed, analyzing methods from yearly LP procedures, to modern closed optimization and new meta-heuristic methods. The review showed that the majority of work is applied to relatively small benchmarks, i.e. an assessment of the performance of the different methods for real world instances was not possible. The meta-heuristic optimization methods are a promising new development, but are not able to guarantee global optima. These methods can nevertheless provide very good solutions (at least for the small benchmarks presented). A problem with these methods is the exhaustive parameter setting necessary for modeling every different application. Furthermore, research on meta-heuristics for irrigation WDN is relatively sparse. Difficulties are

reported in modeling branched irrigation networks, e.g. solution inconsistencies between selected upstream and downstream diameters of the branches' pipelines.

The present dissertation gives a new insight into using MILP approaches for WDN dimensioning. This type of modeling is most suitable for dealing with the discrete nature of the pipeline diameters in dimensioning a WDN. The disadvantage most referred to in the use of MILP for WDN dimensioning was the NP-hard character of such models. In this work it is shown that applying efficient techniques for piecewise approximations to non-linearities, together with modern MILP solvers, can deliver the desired efficient solutions in short computation times. The results of this dissertation further encourage the use of closed mathematical programming methods to address this type of MINLP problem.

2. For the non-linear energy balance constraints (HW-equations) and the energy costs in the objective function, two different linearization approaches were used in this dissertation. The HW equations were linearized using binary variables within a special ordered set of type 1 (SOS1). The power and energy costs were modeled through a piecewise approximation method based on binary variables in a SOS2 framework. The model calculates the required power in each shift defining a network's characteristic operation curve. It was assumed that the determined power can be met by single pumps arranged in parallel, or series combinations. An empirical relationship was derived allowing the estimation of the pump system cost, according to the highest demanded power in all shifts.

3. The model developed was applied to a case-study irrigation settlement in Upper Egypt, to be operated on a rotation water delivery management system. The model shows, with this case-study, that it is possible to determine the optimal size of a WDN together with the optimal dimensioning of the system's components and the optimal management of the irrigation scheduling (shift pattern). The model proved successful

in balancing life-cycle benefits and life-cycle costs, producing coherent optimal designs of the network, allowing informed decisions on the investment's efficiency. More irrigation hydrants will only be included in the optimal solution if their marginal benefit contribution to the objective function is at least as large as their marginal costs. In this process the model accounts for the different elevations of hydrants, i.e. new hydrants are connected sequentially in the direction of higher elevations; the net-benefits of each hydrant are equated to its costs of connection and operation at the different elevations and distances from the pump system.

4. The model is able to optimally determine the shift pattern simultaneously with the size of the network, maximizing the investment's NPV. It derives a shift pattern that best balances the trade-offs between energy and initial investment costs for the given scenario and the required level of IRR. The model produces coherent designs according to scenario expectations, e.g. when diesel costs are more relevant given lower IRR, the model produces designs with larger pipelines inducing lower friction losses, and determines a spatially more concentrated shift pattern. With increasing IRR the weighting of the initial investment costs is emphasized and the model produces network designs that minimize the initial investment costs, selecting pipelines with smaller diameters and determining a more distributed shift pattern.

5. An economic analysis of the design produced for Scenario I and run IRR = 36% was performed. In this context energy and water use indicators were also assessed and compared to a benchmark water distribution network (the CLM irrigation scheme). The purpose of comparison was not only to evaluate the feasibility of the KLB irrigation WDN, but also to evaluate the plausibility of the model's results. It could be shown that the optimization model produced credible results for the proposed energy and water use indicators, e.g. the model calculates in Scenario I (run IRR = 36%) a consumed energy per irrigation supply of 0.32 $\mathrm{kWh \cdot m^{-3}}$ for the KLB system. This is a tenable result when compared with the value of 0.266 $\mathrm{kWh \cdot m^{-3}}$ for the CLM

benchmark. Diesel costs per irrigation supply for this run were calculated as 0.054 $\mathrm{EUR} \cdot \mathrm{m}^{-3}$. All other indicators also showed credible values (see Table 12.6).

6. An analysis was performed in the framework of Scenario II to evaluate the sensitivity of the results to changes in the level of estimated benefits. A high responsiveness of the model-produced WDN designs could be witnessed in the decreasing benefits. There were strong reductions in the level of justifiable investments for the same levels of the demanded IRR. This result is very important: it shows the necessity of investing more effort in a reliable estimation of the project's benefits and also in model improvements leading to more stochastic in farm income estimation.

7. The pipeline and pump system were dimensioned based on peak water demand conditions which produces short irrigation intervals. The model recurrent operation costs (in our case diesel) were modeled differently i.e. based on seasonal average water demands and respective larger irrigation intervals. The calculation of diesel costs should, nevertheless, be based on a more detailed data basis for estimation of total operating hours (e.g. monthly irrigation intervals). Such a detailed calculation could not be done for this model. This difficulty could have led to an overestimation of the operation costs. The model nonetheless shows very plausible results when compared with the CLM benchmark.

8. The model assumes irrigation hydrants composed of 10 farms each, a total of 20 ha per hydrant. These farms should simultaneously operate drip and sprinkler systems in the irrigation shifts. The model can be improved by further dividing each hydrant into smaller specific drip and sprinkler irrigation zones. This could eventually lead to better shift patterns and optimization results, but would increase the model's complexity. The model also predefines the farms' discharges (drip and sprinkler). Introducing these systems' discharges as an extra variable does not seem plausible for the local conditions. The very high infiltration rates of desert soils allow only a

very narrow range of drip and sprinkler discharges.

9. It was assumed in this work that the implementing agency uses governmental funds borrowed in international capital markets at the prevailing interest rate. It was understood that the returns on investment should be at least as large as the capital cost involved. Following the IRR definition, public projects are considered worthwhile if they indicate a positive NPV when discounted at this rate. The model presented uses the internal rate of return (IRR) as an investment decision criterion for the design of the WDN. A formal (mathematical) identity between the concept of IRR and the rate of return calculated by the optimization model was shown.

10. Cost recovery and sustainability of the KLB investment was also addressed for the IRR= 36% design. For this IRR demanded level, the Kalabsha irrigation settlement does not seem to be very sustainable economically. The present study shows in this case that charging farmers only for pumping and non-energy O&M costs would imply an average farm net-return of ca. 28,776 EGP, i.e. the equivalent to only 3,597 EUR per year, or ca. 300 EUR per month. This result is quite conservative, because it does not even include charges for capital cost recovery. The net-return to a family's land, labor and management seems to be very low, and would probably not allow family subsistence.

We can conclude that the present dissertation research was successful in providing a new methodological insight for appraising WDN for irrigation. The model presented extended the dimensioning of WDN to a broader CBA framework. The results are very significant for implementing agencies, because with the proposed model a full economic analysis of the investment is possible. The implementing agency is informed about the optimal size of the irrigation network for given topographical conditions. The agency can decide between implementing one large or several smaller irrigation settlements. Decisions can be guided by the IRR criterion. Complementary development objectives, such as employment, can be used to decide among WDN projects with equal IRR. The present work showed that investments in WDN for irrigation can and should be made under much more comprehensive economic methods and criteria.

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Part V

Appendices

Appendix A

Optimization results for Scenario I (cont.)

Figure A-1: Optimal network size and hydrant schedule for IRR = 32

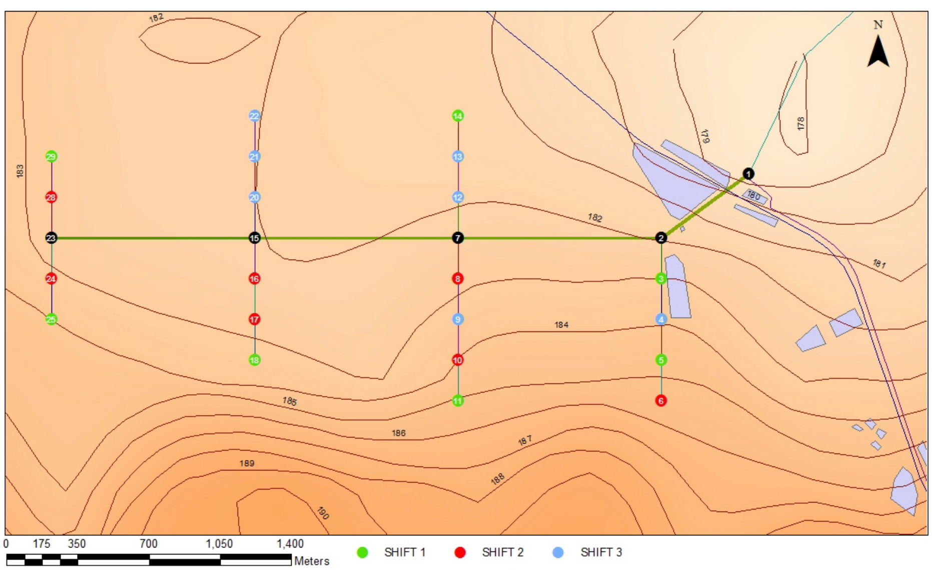


Table A.1: Investment results for optimization run r=32, ef=1, ed=3, and n=30

	EGP	*EUR*
Net present value	1,381,047	172,631
discounted node's net benefits *EGP*	26,661,999	3,332,750
discounted energy costs	15,564,253	1,945,532
discounted OM costs	41,409	5,176
discounted depreciation costs	138,506	17,313
Initial investment costs		
pipeline network	889,700	111,213
pump system	440,283	55,035
farm's irrigation systems	8,206,800	1,025,850
Specific costs per year		
operation and maintenance (OM)	13,254	1,657
depreciation	44,333	5,542

Table A.2: Water and energy use indicators for run r=32, ef=1, ed=3, and n=30

	IRR= 32%	Moreno et al. (2010)	Units
Irrigated area (S_r)	840*	640	ha
Total annual volume of irrigation water supply (V_t)	10,084,435	5,284,266	m^3
Total annual volume of irrigation water supply per irrigated area ($V_tS_r = V_t/S_r$)	23,718	7,429	$m^3 \cdot ha^{-1}$
Average demanded power consumed (DP)	1,502.09	365.4**	kW
Average power per irrigated area ($DPS_r = DP/S_r$)	1.79	0.571***	$kW \cdot ha^{-1}$
Total hours of operation per year ($Thrs$)	2,223.2	na	h
Consumed energy ($CE = DP \cdot Thrs$)	3,339,456	1,406,260	kWh
Consumed energy per irrigation supply ($CEV_t = CE/V_t$)	0.33	0.266	$kWh \cdot m^{-3}$
Consumed energy per irrigated area ($CES_r = CE/S_r$)	3,975.54	2,197	$kWh \cdot ha^{-1}$
Equivalent consumed diesel (ECD)	768,074.8	na	L
Diesel costs per irrigation water supply ($DCV_t = ECD \cdot p_d/V_t$)	0.056	0.0296	$EUR \cdot m^{-3}$
Diesel costs per irrigated area ($DCS_r = ECD \cdot p_d/S_r$)	672	220	$EUR \cdot ha^{-1}$
Benefits per irrigated area ($BS_r = B/S_r$)	$1,230^H$	na	$EUR \cdot ha^{-1}$
Benefits per irrigation water supply ($BV_t = B/V_t$)	0.102	na	$EUR \cdot m^{-3}$

* Two crop seasons

**Average measured absorbed power

***Average measured absorbed power per unit irrigated area

$p_d = 0.735$ EUR $\cdot$ L^{-1}

B = 49,214 EUR multiplied by No. Hydrants in the optimal solution

H = 21 hydrants in the optimal solution

Table A.3: Pipeline's diameters: Optimization run r=32, ef=1, ed=3 and n=30

29-30	0 cm	21-22	44 cm	13-14	31 cm		
28-29	39 cm	20-21	55 cm	12-13	39 cm		
23-28	39 cm	15-20	80 cm	7-12	44 cm		
15-23	62 cm	7-15	80 cm	2-7	100 cm	1-2	120 cm
23-24	44 cm	15-16	44 cm	7-8	44 cm	2-3	44 cm
24-25	44 cm	16-17	39 cm	8-9	44 cm	3-4	39 cm
25-26	0 cm	17-18	35 cm	9-10	44 cm	4-5	44 cm
26-27	0 cm	18-19	0 cm	10-11	39 cm	5-6	35 cm

Table A.4: Node's pressures for run: r=32, ef=1, ed=3, and n=30

	30	0 m	22	0 m	14	41.6 m		
	29	40.5 m	21	0 m	13	44 m		
	28	41.3 m	20	0 m	12	44.8 m	1	49.5 m
SHIFT 1	23	42.1 m	15	44.1 m	7	45.1 m	2	46.2 m
	24	41.2 m	16	43.7 m	8	44 m	3	43.5 m
	25	40 m	17	42.4 m	9	43 m	4	41.8 m
	26	0 m	18	40.4 m	10	41.9 m	5	40.7 m
	27	0 m	19	0 m	11	40.1 m	6	0 m
	30	0 m	22	0 m	14	0 m		
	29	0 m	21	0 m	13	0 m		
	28	40.1 m	20	0 m	12	0 m	1	49.3 m
SHIFT 2	23	40.8 m	15	42.9 m	7	44.5 m	2	45.9 m
	24	40 m	16	41.3 m	8	42.3 m	3	44.4 m
	25	0 m	17	40 m	9	41.3 m	4	42.7 m
	26	0 m	18	0 m	10	40.2 m	5	41.6 m
	27	0 m	19	0 m	11	0 m	6	40.1 m
	30	0 m	22	40 m	14	0 m		
	29	0 m	21	40.5 m	13	40 m		
	28	0 m	20	41 m	12	40.8 m	1	46.9 m
SHIFT 3	23	0 m	15	41.2 m	7	42.2 m	2	43.6 m
	24	0 m	16	0 m	8	41.1 m	3	42.1 m
	25	0 m	17	0 m	9	40.1 m	4	40.4 m
	26	0 m	18	0 m	10	0 m	5	0 m
	27	0 m	19	0 m	11	0 m	6	0 m

Table A.5: Pipeline's discharges in L/s for the run r=32 ef=1 ed=3 and n=30

SHIFT 1							
29-30	0 L/s	21-22	0 L/s	13-14	180 L/s		
28-29	180 L/s	20-21	0 L/s	12-13	180 L/s		
23-28	180 L/s	15-20	0 L/s	7-12	180 L/s		
15-23	360 L/s	7-15	540 L/s	2-7	900 L/s	1-2	1260 L/s
23-24	180 L/s	15-16	180 L/s	7-8	180 L/s	2-3	360 L/s
24-25	180 L/s	16-17	180 L/s	8-9	180 L/s	3-4	180 L/s
25-26	0 L/s	17-18	180 L/s	9-10	180 L/s	4-5	180 L/s
26-27	0 L/s	18-19	0 L/s	10-11	180 L/s	5-6	0 L/s
SHIFT 2							
29-30	0 L/s	21-22	0 L/s	13-14	0 L/s		
28-29	0 L/s	20-21	0 L/s	12-13	0 L/s		
23-28	180 L/s	15-20	0 L/s	7-12	0 L/s		
15-23	360 L/s	7-15	720 L/s	2-7	1080 L/s	1-2	1260 L/s
23-24	180 L/s	15-16	360 L/s	7-8	360 L/s	2-3	180 L/s
24-25	0 L/s	16-17	180 L/s	8-9	180 L/s	3-4	180 L/s
25-26	0 L/s	17-18	0 L/s	9-10	180 L/s	4-5	180 L/s
26-27	0 L/s	18-19	0 L/s	10-11	0 L/s	5-6	180 L/s
SHIFT 3							
29-30	0 L/s	21-22	180 L/s	13-14	0 L/s		
28-29	0 L/s	20-21	360 L/s	12-13	180 L/s		
23-28	0 L/s	15-20	540 L/s	7-12	360 L/s		
15-23	0 L/s	7-15	540 L/s	2-7	1080 L/s	1-2	1260 L/s
23-24	0 L/s	15-16	0 L/s	7-8	180 L/s	2-3	180 L/s
24-25	0 L/s	16-17	0 L/s	8-9	180 L/s	3-4	180 L/s
25-26	0 L/s	17-18	0 L/s	9-10	0 L/s	4-5	0 L/s
26-27	0 L/s	18-19	0 L/s	10-11	0 L/s	5-6	0 L/s

Appendix B

Optimization results for Scenario II (cont.)

Figure B-1: Optimal network size and hydrant schedule for IRR = 16

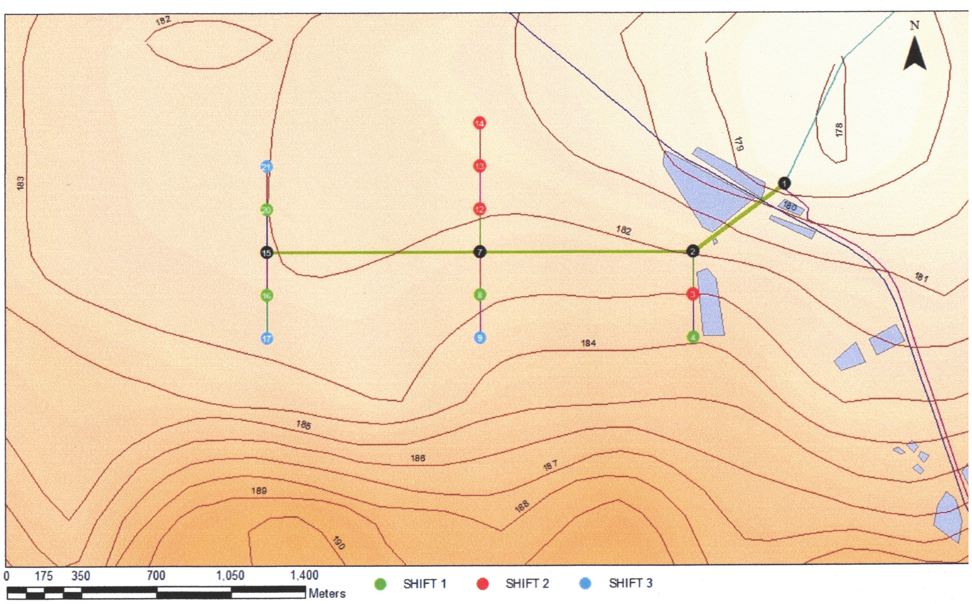


Table B.1: Investment results for optimization run r=16, ef=1, ed=3, and n=30

	EGP	*EUR*
Net present value	473,388	59,173
discounted node's net benefits *EGP*	22,735,019	2,841,877
discounted energy costs	16,924,861	2,115,608
discounted OM costs	47,418	5,927
discounted depreciation costs	169,135	21,142
Initial investment costs		
pipeline network	583,470	72,934
pump system	237,947	29,743
farm's irrigation systems	4,298,800	537,350
Specific costs per year		
operation and maintenance (OM)	7,676	960
depreciation	27,381	3,423

Table B.2: Water and energy use indicators for run r=16, ef=1, ed=3, and n=30

	IRR= 16%	Moreno et al. (2010)	Units
Irrigated area (S_r)	440*	640	ha
Total annual volume of irrigation water supply (V_t)	5,282,323	5,284,266	m^3
Total annual volume of irrigation water supply per irrigated area ($V_tS_r = V_t/S_r$)	12,005	7,429	$m^3 \cdot ha^{-1}$
Average system's power consumed (SP)	753.08	365.4**	kW
Average system's power consumed per irrigated area ($SPS_r = SP/S_r$)	1.71	0.571***	$kW \cdot ha^{-1}$
Total hours of operation per year ($Thrs$)	2,223.2	na	h
Consumed energy ($CE = SP \cdot Thrs$)	1,674,247	1,406,260	kWh
Consumed energy per irrigation supply ($CEV_t = CE/V_t$)	0.32	0.266	$kWh \cdot m^{-3}$
Consumed energy per irrigated area ($CES_r = CE/S_r$)	3,805.1	2,197	$kWh \cdot ha^{-1}$
Equivalent consumed diesel (ECD)	385,076.5	na	L
Diesel costs per irrigation water supply ($DCV_t = ECD \cdot p_d/V_t$)	0.054	0.0296	$EUR \cdot m^{-3}$
Diesel costs per irrigated area ($DCS_r = ECD \cdot p_d/S_r$)	643.25	220	$EUR \cdot ha^{-1}$
Benefits per irrigated area ($BS_r = B/S_r$)	1230^H	na	$EUR \cdot ha^{-1}$
Benefits per irrigation water supply ($BV_t = B/V_t$)	0.102	na	$EUR \cdot m^{-3}$

* Two crop seasons

**Average measured absorbed power

***Average measured absorbed power per unit irrigated area

$p_d = 0.735$ EUR $\cdot$ L^{-1}

B = 49,214 EUR multiplied by No. Hydrants in the optimal solution

H = 11 hydrants in the optimal solution

Table B.3: Pipeline's diameters: Optimization run r=16, ef=1, ed=3 and n=30

29-30	0 cm	21-22	0 cm	13-14	44 cm		
28-29	0 cm	20-21	39 cm	12-13	62 cm		
23-28	0 cm	15-20	44 cm	7-12	80 cm		
15-23	0 cm	7-15	62 cm	2-7	80 cm	1-2	80 cm
23-24	0 cm	15-16	44 cm	7-8	39 cm	2-3	39 cm
24-25	0 cm	16-17	44 cm	8-9	39 cm	3-4	44 cm
25-26	0 cm	17-18	0 cm	9-10	0 cm	4-5	0 cm
26-27	0 cm	18-19	0 cm	10-11	0 cm	5-6	0 cm

Table B.4: Node's pressures for run: r=16, ef=1, ed=3, and n=30

	30	0 m	22	0 m	14	0 m		
	29	0 m	21	0 m	13	0 m		
	28	0 m	20	40 m	12	0 m	1	47.1 m
SHIFT 1	23	0 m	15	40.4 m	7	42 m	2	43.2 m
	24	0 m	16	40 m	8	40.6 m	3	41.3 m
	25	0 m	17	0 m	9	0 m	4	40 m
	26	0 m	18	0 m	10	0 m	5	0 m
	27	0 m	19	0 m	11	0 m	6	0 m
	30	0 m	22	0 m	14	40 m		
	29	0 m	21	0 m	13	40.4 m		
	28	0 m	20	0 m	12	40.7 m	1	45.8 m
SHIFT 2	23	0 m	15	0 m	7	40.7 m	2	42 m
	24	0 m	16	0 m	8	0 m	3	40.1 m
	25	0 m	17	0 m	9	0 m	4	0 m
	26	0 m	18	0 m	10	0 m	5	0 m
	27	0 m	19	0 m	11	0 m	6	0 m
	30	0 m	22	0 m	14	0 m		
	29	0 m	21	40.2 m	13	0 m		
	28	0 m	20	40.9 m	12	0 m	1	47.7 m
SHIFT 3	23	0 m	15	41.4 m	7	42.9 m	2	44.2 m
	24	0 m	16	40.9 m	8	41.5 m	3	0 m
	25	0 m	17	40 m	9	40.2 m	4	0 m
	26	0 m	18	0 m	10	0 m	5	0 m
	27	0 m	19	0 m	11	0 m	6	0 m

Table B.5: Pipelines discharge's in L/s for the run r=16 ef=1 ed=3 and n=30

SHIFT 1							
29-30	0 L/s	21-22	0 L/s	13-14	180 L/s		
28-29	180 L/s	20-21	0 L/s	12-13	180 L/s		
23-28	180 L/s	15-20	0 L/s	7-12	180 L/s		
15-23	360 L/s	7-15	540 L/s	2-7	900 L/s	1-2	1260 L/s
23-24	180 L/s	15-16	180 L/s	7-8	180 L/s	2-3	360 L/s
24-25	180 L/s	16-17	180 L/s	8-9	180 L/s	3-4	180 L/s
25-26	0 L/s	17-18	180 L/s	9-10	180 L/s	4-5	180 L/s
26-27	0 L/s	18-19	0 L/s	10-11	180 L/s	5-6	0 L/s
SHIFT 2							
29-30	0 L/s	21-22	0 L/s	13-14	0 L/s		
28-29	0 L/s	20-21	0 L/s	12-13	0 L/s		
23-28	180 L/s	15-20	0 L/s	7-12	0 L/s		
15-23	360 L/s	7-15	720 L/s	2-7	1080 L/s	1-2	1260 L/s
23-24	180 L/s	15-16	360 L/s	7-8	360 L/s	2-3	180 L/s
24-25	0 L/s	16-17	180 L/s	8-9	180 L/s	3-4	180 L/s
25-26	0 L/s	17-18	0 L/s	9-10	180 L/s	4-5	180 L/s
26-27	0 L/s	18-19	0 L/s	10-11	0 L/s	5-6	180 L/s
SHIFT 3							
29-30	0 L/s	21-22	180 L/s	13-14	0 L/s		
28-29	0 L/s	20-21	360 L/s	12-13	180 L/s		
23-28	0 L/s	15-20	540 L/s	7-12	360 L/s		
15-23	0 L/s	7-15	540 L/s	2-7	1080 L/s	1-2	1260 L/s
23-24	0 L/s	15-16	0 L/s	7-8	180 L/s	2-3	180 L/s
24-25	0 L/s	16-17	0 L/s	8-9	180 L/s	3-4	180 L/s
25-26	0 L/s	17-18	0 L/s	9-10	0 L/s	4-5	0 L/s
26-27	0 L/s	18-19	0 L/s	10-11	0 L/s	5-6	0 L/s

Appendix C

The whole model set-up

List of sets

i = set of pipe's entry edges

j = set of pipe's ending edges

A = set of all pipe Sections (Sections with and without irrigation nodes)

H = set of pipe Sections ending in an irrigation node for j

s = set of irrigation shifts

d = set for setting discharge multiples

k = set of commercial pipe diameters

a = set for grid points in linear approximations

b = set for grid points in linear approximations

List of parameters

r = interest rate

e_f = escalation rate of investment's benefits

e_d = escalation rate of energy prices

p_d = price of diesel

n = life time of the project

$z\,(i)$ = elevation at pipe's edge i

$z\,(j)$ = elevation at pipe's edge j

$l\,(ij)$ = the pipe length at section ij

$C_{hw}\,(k)$ = the Hazen-Williams coefficient of roughness for pipe type k

$diam\,(k)$ = the diameter of pipe k

$pipeprice\,(k)$ = pipe price for type k

q_H = the hydrant discharge

$val\,(dd)$ = represents parameter constructing multiples of set dd

$maxQ$ = maximal discharge at the system's pump

$maxP$ = maximal pressure at the system's pump

$P_{\min}$ = minimum demanded pressure for each irrigating hydrant

η = combined power system efficiency

$pointsa\,(a)$ = grid points (break points) in the domain $0 \leq y_1 \leq 1$ of Y_1

$pointsb\,(b)$ = grid points in the domain $-\frac{1}{2} \leq y_2 \leq \frac{1}{2}$ of Y_2

List of decision variables

NPV = net present worth of the investment, model's objective variable

$B(i,j)$ = net benefit of irrigated agriculture for each node

$PW(s)$ = power developed by the system's pump in shift s

EC = yearly energy costs of the pumping system

$Q(s,ij)$ = discharge in shift s in pipe Section ij

$P(s,i)$ = pressure at pipe's edge i in shift s

$P(s,j)$ = pressure at pipe's edge j in shift s

$\overline{QP}$ = linear approximation of the bilinear product $Q \cdot P$

PC = pipe costs of all sections of the network

$V(s,j)$ = binary variable indicating if the hydrant is open or closed in shift s

$I(ij,s,d,k)$ = binary variable choice in section (i,j) of discharges d and pipe types k

$NODE(ij,k)$ = binary variable indicating the chosen pipe type k for section (i,j)

Y_1 = auxiliar variable for linear approximation of the power function

Y_2 = auxiliar variable for linear approximation of the power function

$A(s,a)$ = SOS2 ordered set variable for weighting in the linear approximations to the power and energy cost Equations

$B(s,b)$ = SOS2 ordered set variable for weighting in the linear approximations to the power and energy cost Equations

The objective function

$$
\begin{aligned}
NPV \; = \; & \sum_{ij \,\in\, H} \sum_{k} return \cdot NODE\left(ij,k\right) \cdot \left[\frac{(1+r)^{n} - (1+e_{f})^{n}}{(1+r) - (1+e_{f})} \cdot \frac{1}{(1+r)^{n}} \right] \qquad \text{(C.1)} \\
& - \sum_{s} \Phi \cdot \left[\overline{QP}\left(s\right) \cdot maxQ \cdot maxP \right] \cdot \left[\frac{(1+r)^{n} - (1+e_{d})^{n}}{(1+r) - (1+e_{d})} \cdot \frac{1}{(1+r)^{n}} \right] \\
& - \sum_{ij \,\in\, A} \sum_{k} pipecost(k) \cdot l(ij) \cdot NODE(ij,k) \cdot \left[1 + \left(0.005 + \frac{1}{n} \right) \cdot \frac{(1+r)^{n} - 1}{r\,(1+r)^{n}} \right] \\
& - 285 \cdot PW\left(\text{"1"}\right) \cdot \left[1 + \left(0.02 + \frac{1}{n} \right) \cdot \frac{(1+r)^{n} - 1}{r\,(1+r)^{n}} \right] \\
& - \sum_{ij \,\in\, N} \sum_{k} nodecost \cdot NODE(ij,k)
\end{aligned}
$$

were, $\Phi = \left[\frac{\rho g}{\eta} \cdot \left(T_{sh} \cdot \left(\frac{1}{6} + \frac{1}{10} \right) \cdot 280 \right) \cdot cf \cdot p_{d} \right]$ as discussed in Chapter 7.

Equation block 1: Law of energy conservation

$$
\begin{gathered}
P\left(s,i\right) - P\left(s,j\right) + \left[z(i) - z(j)\right] = \qquad \text{(C.2)} \\
\sum_{d} \sum_{k} l(ij) \cdot 10.68 \cdot diam\left(k\right)^{-4.87} \cdot \left(\frac{val\left(d\right) \cdot q_{N}}{C_{hw}\left(k\right)} \right)^{1.852} \cdot I\left(ij,s,d,k\right), \;\forall\; \left(s,ij\right)
\end{gathered}
$$

Equation block 2: The minimum pressure at the irrigation nodes

$$
P\left(s,i_{h}\right) \geq 40 \cdot V\left(s,i_{h}\right) \;\; \forall \; s \text{ and } i_{h} \in H \qquad \text{(C.3)}
$$

Equation block 3: The pump's power and energy cost Equations

Linear approximations to the power and energy cost functions

$$
Y1(s) = \left[\frac{Q(s,\text{"1"},\text{"2"})}{1000 \cdot \max Q} + \frac{P(s,\text{"1"})}{\max H} \right] \cdot \frac{1}{2} \qquad \text{(C.4)}
$$

$$
Y2(s) = \left[\frac{Q(s,\text{"1"},\text{"2"})}{1000 \cdot \max Q} - \frac{P(s,\text{"1"})}{\max H} \right] \cdot \frac{1}{2} \qquad \text{(C.5)}
$$

$$Y_1(s) = \sum_{a} A(s,a) \cdot pointsa(a), \ \forall \ s \tag{C.6}$$

$$Y_2(s) = \sum_{gb} B(s,b) \cdot pointsb(b), \ \forall \ s \tag{C.7}$$

$$\sum_{a} A(s,a) = 1, \ \forall \ s \tag{C.8}$$

$$\sum_{b} B(s,b) = 1, \ \forall \ s \tag{C.9}$$

$$Y_1^2(s) = \sum_{ga} A(s,a) \cdot (pointsa(a))^2, \ \forall \ s \tag{C.10}$$

$$Y_2^2(s) = \sum_{b} B(s,pb) \cdot (pointsb(b))^2, \ \forall \ s \tag{C.11}$$

$$\overline{QP}(s) = Y_1^2(s) - Y_2^2(s) = \sum_{a} A(s,a) \cdot (pointsa(a))^2 - \sum_{b} B(s,b) \cdot (pointsb(b))^2, \ \forall \ s \tag{C.12}$$

The power function

$$PW\left(s\right)=\frac{\rho g\cdot\ \left[\left(\overline{QP}\right)\left(s\right)\ \cdot maxQ\cdot maxP\right]}{\eta\cdot 1000}\ \text{in}\ \left[kW\right],\ \forall\ s \tag{C.13}$$

Energy costs function

$$EC=\left[T_{sh}\cdot(\frac{1}{6}+\frac{1}{10})\cdot 140\right]\cdot\left(\sum_{s}PW(s)\cdot c_{f}\right)\cdot p_{d}\ \text{in}\ \left[EGP\right] \tag{C.14}$$

Equation block 4: Pipeline costs

$$\sum_{ij\in A}PC=\sum_{ij}\sum_{k}pprice\left(k\right)\ \cdot l\left(ij\right)\ \cdot NODE(ij,k) \tag{C.15}$$

Equation block 5: Velocity limits for pipes' water flow

$$\sum_{d}\sum_{k}\left(\frac{val\left(d\right)\ \cdot q_{H}}{\pi\left(\frac{diam(k)}{2}\right)^{2}}\cdot I\left(ij,s,dd,k\right)\right)\leq 3\ \forall\ (i,j),s \tag{C.16}$$

$$\sum_{d}\sum_{k}\left(\frac{val\left(d\right)\ \cdot q_{H}}{\pi\left(\frac{diam(k)}{2}\right)^{2}}\cdot I\left(ij,s,dd,k\right)\right)\geq 1.5\cdot\sum_{d}\sum_{k}I(ij,s,d,k)\ \forall\ (i,j),s \tag{C.17}$$

Equation block 6: Law of conservation of mass for the main distribution pipes in the network

$$Q\left(s,\text{“1”},\text{“2”}\right)=Q\left(s,\text{“2”},\text{“3”}\right)+Q\left(s,\text{“2”},\text{“7”}\right) \tag{C.18}$$

$$Q(s,\text{“2”},\text{“7”})=Q(s,\text{“7”},\text{“12”})+Q(s,\text{“7”},\text{“8”})+Q(s,\text{“7”},\text{“15”}) \tag{C.19}$$

$$Q(s,\text{“7”},\text{“15”})=Q(s,\text{“15”},\text{“20”})+Q(s,\text{“15”},\text{“16”})+Q(s,\text{“15”},\text{“23”}) \tag{C.20}$$

$$Q(s,\text{“15”},\text{“23”})=Q(s,\text{“23”},\text{“24”})+Q(s,\text{“23”},\text{“28”}) \tag{C.21}$$

Equation block 7: Law of conservation of mass for network's branches

$$Q\left(s,ij\right)=\sum_{\overline{ij}(ij)}V(s,\overline{j})\cdot q_{H},\quad\ \forall\ s\ \text{and}\ ij\in I \tag{C.22}$$

$$Q\,(s,ij) = \sum_{\overline{ij}(ij)} V(s,\overline{j}) \cdot q_H, \quad \forall\ s \text{ and } ij \in II \tag{C.23}$$

$$Q\,(s,ij) = \sum_{\overline{ij}(ij)} V(s,\overline{j}) \cdot q_H, \quad \forall\ s \text{ and } ij \in III \tag{C.24}$$

$$Q\,(s,ij) = \sum_{\overline{ij}(ij)} V(s,\overline{j}) \cdot q_H, \quad \forall\ s \text{ and } ij \in IV \tag{C.25}$$

$$Q\,(s,ij) = \sum_{\overline{ij}(ij)} V(s,\overline{j}) \cdot q_H, \quad \forall\ s \text{ and } ij \in V \tag{C.26}$$

$$Q\,(s,ij) = \sum_{\overline{ij}(ij)} V(s,\overline{j}) \cdot q_H, \quad \forall\ s \text{ and } ij \in VI \tag{C.27}$$

$$Q\,(s,ij) = \sum_{\overline{ij}(ij)} V(s,\overline{j}) \cdot q_H, \quad \forall\ (s \text{ and } ij \in VII) \tag{C.28}$$

Other constraints and model equations

Discharges in the pipelines

$$\sum_{d} I(ij,s,d,k) \leq NODE(ij,k),\ \forall\ ij,\ s,\ k. \tag{C.29}$$

Number of pipelines per section

$$\sum_{k} NODE(ij,k) \leq 1\ \forall\ (i,j) \tag{C.30}$$

Number of times a hydrant irrigates

$$\sum_{k} NODE(ij,k) = \sum_{s} V(s,j),\ \forall\ s,\ ij,\ \text{and } j \notin \{2,7,15,23\}\,. \tag{C.31}$$

The pump unit is priced using the highest demanded power

$$PW("1") \geq PW("2") \tag{C.32}$$

$$PW("2") \geq PW("3") \tag{C.33}$$

Appendix D

Programming the model using GAMS

D.1 Introduction

The General Algebraic Modeling System (GAMS) is a special software for modeling linear (LP), nonlinear (NLP) and mixed integer (MILP) optimization problems. The GAMS system is especially designed for modelling large, complex problems. For more information see www.gams.com. The model of this disseration used the CPLEX solver integrated in GAMS. According to ILOG, Inc. (2013):

> "CPLEX is a solver for linear, mixed-integer and quadratic programming problems developed by ILOG (http://www.ilog.com/products/cplex/). CPLEX contains a primal simplex algorithm, a dual simplex algorithm, a network optimizer, an interior point barrier algorithm, a mixed integer algorithm and a quadratic capability. CPLEX also contains an infeasibility finder. For problems with integer variables, CPLEX uses a branch and bound algorithm (with cuts) and supports specially ordered set variables SOS1, SOS2 as well as semi-continuous and semi-integer variables. Base CPLEX solves LP and RMIP model types. Additional capabilities of CPLEX can be licensed involving Barrier, MIP and QCP capability".

D.2 The GAMS code

Sets

i start edge of a pipeline/ 1*30 /

j end edge of a pipeline/ 1*30 /

ij(*i*, *j*) suitable combination of edges - multi-dimensional set - many-to-many mapping/1 .2, 2 .3, 3 .4, 4 .5, 5 .6, 2 .7, 7 .8, 8 .9, 9 .10, 10 .11, 7 .12, 12 .13, 13 .14, 7 .15, 15 .16, 16 .17, 17 .18, 18 .19, 15 .20, 20 .21, 21 .22, 15 .23, 23 .24, 24 .25, 25 .26, 26 .27, 23 .28, 28 .29, 29 .30/

ib(*i*) the irrigation node's set (subset of i)/ 3, 4, 5, 6, 8, 9, 10, 11, 12, 13, 14, 16, 17, 18, 19, 20, 21, 22, 24, 25, 26, 27, 28, 29, 30 /

k different piline types/27-5 ,31-0,34-9, 39-4, 44-3, 49-2, 55-1, 62-0, 69-8, 80-0,100, 120 /

d set for discharges/1*10/

s set for the irrigation shifts/1*3/;

a pice-wise linear approx. variable A(s,sa)/1*9/

b pice-wise linear approx. variable B(s,sa)/1*9/

Scalars

Chw Hazen-Williams coefficient for PVC pipes roughness / 150 /

discharge discharge per node in l per s /180/

r discount rate /0.36/

ef escalation rate /0.01/

ed escalation rate /0.03/

n life time of the project /30/;

Parameters

diam(*k*) diameters for different pipeline types / 27-5 0.275, 31-0 0.310, 34-9 0.349, 39-4 0.394, 44-3 0.443, 49-2 0.492, 55-1 0.551, 62-0 0.620, 69-8 0.698, 80-0 0.800, 100 1, 120 1.20 /;

zi(*i*) elevation at edge i/1 178.9, 2 181.9, 3 183, 4 183.9, 5 184.6, 6 185.4, 7 182.2, 8 182.8, 9 183.4, 10 184, 11 185.1, 12 182, 13 182, 14 182, 15 182.2, 16 182.2, 17 182.7, 18 183.3, 19 185, 20 182.2, 21 182.2, 22 182.2, 23 182.7, 24 183.1, 25 183.9, 26 184.3, 27 184.5, 28 182.7, 29 182.7, 30 185 /;

zj(*j*) elevation at edge j/ 1= 178.9, 2= 181.9, 3= 183, 4= 183.9, 5= 184.6, 6= 185.4, 7= 182.2, 8= 182.8, 9= 183.4, 10= 184, 11= 185.1, 12= 182, 13= 182, 14= 182, 15= 182.2, 16= 182.2, 17= 182.7, 18= 183.3, 19= 185, 20= 182.2, 21= 182.2, 22= 182.2, 23= 182.7, 24= 183.1, 25= 183.9, 26= 184.3, 27= 184.5, 28= 182.7, 29= 182.7, 30= 185 /;

arc(*i, j*) lenght of pipeline in section ij/1 .2 533, 2 .3 200, 3 .4 200, 4 .5 200, 5 .6 100, 2 .7 1000, 7 .8 200, 8 .9 200, 9 .10 200, 10 .11 200, 7 .12 200, 12 .13 200, 13 .14 200, 7 .15 1000, 15 .16 200, 16 .17 200, 17 .18 200, 18 .19 200, 15 .20 200, 20 .21 200, 21 .22 200, 15 .23 1000, 23 .24 200, 24 .25 200, 25 .26 200, 26 .27 200, 23 .28 200, 28 .29 200, 29 .30 200 / ;

pprice(*k*) price of pipeline type k/27-5 27.99, 31-0 34.02 , 34-9 43.92, 39-4 54.90 , 44-3 68.85 , 49-2 84.42 , 55-1 106.20 , 62-0 132.30 ,69-8 170.10 , 80-0 180, 100 190, 120 200 / ;

$bpointsa$ breakpoint sets for pice-wise linear approximations/1 0, 2 0.125, 3 0.250, 4 0.375, 5 0.5, 6 0.625, 7 0.75, 8 0.875, 9 1 /

$bpointsb$ breakpoint sets for pice-wise linear approximations/1 -0.5, 2 -0.375, 3 -0.25, 4 -0.125, 5 0, 6 0.125, 7 0.25, 8 0.375, 9 0.5/;

$bpointsa2$ breakpoint sets for pice-wise linear approximations/1 0, 2 0.015625, 3 0.0625, 4 0.140625, 5 0.25, 6 0.390625 , 7 0.5625 , 8 0.765625 , 9 1/ ;

$bpointsb2$ breakpoint sets for pice-wise linear approximations/1 0.25, 2 0.140625, 3 0.0625, 4 0.015625, 5 0, 6 0.015625, 7 0.0625, 8 0.140625, 9 0.25/ ;

$area$ agricultural area in each irrigation node/20/

D_h available hours a day for irrigation /20/

F irrigation interval (days) /3/

$\max Q$ maximal discharge at the pump for all shifts/2/

$\max P$ maximal pressure at the pump for all shifts/70/

Parameters

$val(d)$ extracts different values according to the set d for producing discharge multiples;

$val(d)$ = ord(d);

PVe_f discount factor with escalation rates;

PVe_f = (((1+ef)**n - (1+r)**n)/((1+ef)-(1+r)))*(1/((1+r)**n))

PVe_d discount factor without escalation rates;

PVe_d = (((1+ed)**n - (1+r)**n)/((1+ed)-(1+r)))*(1/((1+r)**n))

PVe discount factor without escalation rates;

```
PV = ((1+r)**n - 1)/(r*(1+r)**n) ;

Parameters return potential return per irrigation node (base year 2010) ;

loop(ij(i,j)$((ord(j) ne 2) and (ord(j) ne 7) and (ord(j) ne 15) and (ord(j) ne 23)),
    return(ij) = 393710)

loop(ij(i,j)$((ord(j) eq 2) and (ord(j) eq 7) and (ord(j) eq 15) and (ord(j) eq 23)),
    return(ij) = 0);

parameters costnode cost of implementing drip and sprinkler irrigation systems per node
    (base year 2010) ;

loop(ij(i,j)$((ord(j) ne 2) and (ord(j) ne 7) and (ord(j) ne 15) and (ord(j) ne 23)),
    costnode(ij) = 390800)

loop(ij(i,j)$((ord(j) eq 2) and (ord(j) eq 7) and (ord(j) eq 15) and (ord(j) eq 23)),
    costnode(ij) = 0 )

******************************************************************************

Variables NPV, Y2;

Positive variables Q, EC, PC, Y1, qh, PW, P

Binary variables V, IN, NODE

SOS2 variables A, B ;
```

****

EQUATIONS

****

Equation eq_NPV the objective function ;

eq_NPV..

NPV =E= SUM(ij, SUM(k, $return(ij)$*$NODE(ij,k)$))*PVe_f - EC*PVe_d

- SUM(ij, SUM(k, $pprice(k)$*$arc(ij)$*$NODE(ij,k)$))*(1 + 0.005*PV + (1/n)*PV)

- SUM(ij, $costnode(ij)$*SUM(k, $NODE(ij,k)$)) - 287.6*PW("1")*(1+0.02*PV + (1/n)*PV);

Equation NODE_no assures that only one pipeline is possible for connecting each node;

NODE_no(ij)..

SUM(k, $NODE(ij,k)$) =L=1 ;

****

Equation block 1: Law of energy conservation

****

Equation energy_conservation;

energy_conservation(s, ij(i,j))..

$P(s,i) - P(s,j) + (zi(i) - zj(j))$*SUM(d$(ijd(ij,d)), SUM(k, IN(ij,s,d,k))) =E=

SUM(d$(ijd(ij,d)), SUM(k, $arc(ij)$*10.68*($diam(k)$**(-4.87))

((((1/1000)$val(d)$*$discharge$)/Chw)**1.852)*$IN(ij,s,d,k)$)) ;

```
*******************************************************************************
* Equation block 2: Minimum operating pressures at the irrigation nodes
*******************************************************************************
Equation pressure_min;
pressure_min(s,ib)..
P(s,ib) =G= 40*V(s,ib)
;
*******************************************************************************
* Equation block 3: The pump's power and energy cost equations
*******************************************************************************
* Equations for pice-wise linear approximations (see Chapter 9)

Equation eq_Y1;
eq_Y1(s)..
Y1(s)=E= ((Q(s,"1","2")/1000)/maxQ + P(s,"1")/maxH)/2
;
Equation eq_Y2;
eq_Y2(s)..
Y2(s) =E= ((Q(s,"1","2")/1000)/maxQ - P(s,"1")/maxH)/2
;
Equation eq_Y1A;
eq_Y1A(s)..
Y1(s) =E= SUM(sa, A(s,sa)*bpointsa(sa))
;
Equation eq_Y2B;
eq_Y2B(s)..
Y2(s) =E= SUM(sb, B(s,sb)*bpointsb(sb))
;
```

```
Equation eq_A;
eq_A(s)..
SUM(sa, A(s,sa))=E= 1
;
Equation eq_B;
eq_B(s)..
SUM(sb, B(s,sb))=E= 1
;
Equation eq_powerY1Y2;
eq_powerY1Y2(s)..
QH(s)=E= SUM(sa, A(s,sa)*bpointsa2(sa)) - SUM(sb, B(s,sb)*bpointsb2(sb))
;
Equation eq_powerQHP;
eq_powerQHP(s)..
PW(s) =E= 9.81*(QH(s)*maxQ*maxH)
;
*********
* The following two sets of equations force the higst power consumption to be in shift one.
* The identification allows the estimation of the price pump dependent on the highest power demand
*********
Equation eq_powerQH_1 ;
eq_powerQH_1..
PW("1") =G= PW("2")
;
```

```
Equation eq_powerQHP_2;
eq_powerQHP_2..
PW("2") =G= PW("3")
;
***********
* The total energy costs: sum of the energy costs per irrigation shift
***********
Equation eq_energycosts;
eq_energycosts..
EC =E= 19.85*(1/6 +1/10)*140*SUM(s, PW(s))*0.23*5.88
;
*********************************************************************************
* Equation block 4: Pipeline costs
*********************************************************************************
Equation eq_PIPECOSTS;
eq_PIPECOSTS..
PC =E= SUM(ij, SUM(k, pprice(k) * arc(ij) * NODE(ij,k)))

*********************************************************************************
* Equation block 5: Velocity limits for water flow
*********************************************************************************
Equation speed_limit_u upper water velocity limit;
speed_limit_u(s, ij(i,j))..
SUM(d$(ijd(ij,d)), SUM(k,((1/1000)*val(d)*discharge*IN(ij,s,d,k))/(3.14*((diam(k)/2)**2))))
    =L= 3;
```

Equation speed_limit_d lower water velocity limit;

speed_limit_d(s, ij(i,j))..

SUM(d$(ijd(ij,d)), SUM(k,((1/1000)$*val(d)*discharge*IN(ij,s,d,k)$)/(3.14*(($diam(k)$/2)**2))))
=G= 1.5*SUM(d$(ijd(ij,d)), SUM(k,$IN(ij,s,d,k)$));

****

* Equation block 6: Law of mass conservation - Discharge balances in the main distribution pipelines

****

Equation eq_discharge;
eq_discharge(s,ij(i,j))..
$Q(s,ij)$ =E=SUM(d$(ijd(ij,d)), SUM(k, $val(d)*discharge*IN(ij,s,d,k)$))
;
Equation flowbalance1;
flowbalance1(s)..
$Q(s,"1","2")$ =E= $Q(s,"2","3") + Q(s,"2","7")$
;
Equation flowbalance2;
flowbalance2(s)..
$Q(s,"2","7")$ =E= $Q(s,"7","12") + Q(s,"7","8") + Q(s,"7","15")$
;
Equation flowbalance3;
flowbalance3(s)..
$Q(s,"7","15")$ =E= $Q(s,"15","20") + Q(s,"15","16") + Q(s,"15","23")$
;

Equation flowbalance4;

flowbalance4(s)..

$Q(s,"15","23")$ =E= $Q(s,"23","24") + Q(s,"23","28")$

;

****

* Equation block 7: Law of mass conservation - Discharge balances in the network branches

****

Equation flowbalance5;

flowbalance5(s)..

$Q(s,"2","3")$ =E= $(V(s,"3") + V(s,"4") + V(s,"5") + V(s,"6")) * discharge$

;

Equation flowbalance6;

flowbalance6(s)..

$Q(s,"3","4")$ =E= $(V(s,"4") + V(s,"5") + V(s,"6")) * discharge$

;

Equation flowbalance7;

flowbalance7(s)..

$Q(s,"4","5")$ =E= $(V(s,"5") + V(s,"6")) * discharge$

;

Equation flowbalance8;

flowbalance8(s)..

$Q(s,"5","6")$ =E= $V(s,"6") * discharge$

;

Equation flowbalance9;

flowbalance9(s)..

$Q(s,"7","8")$ =E= $(V(s,"8") + V(s,"9") + V(s,"10") + V(s,"11")) * discharge$

;

Equation flowbalance10;

flowbalance10(s)..

$Q(s,"8","9")$ =E= $(V(s,"9") + V(s,"10") + V(s,"11")) * discharge$

;

Equation flowbalance11;

flowbalance11(s)..

$Q(s,"9","10")$ =E= $(V(s,"10") + V(s,"11")) * discharge$

;

Equation flowbalance12;

flowbalance12(s)..

$Q(s,"10","11")$ =E= $V(s,"11") * discharge$

;

Equation flowbalance13;

flowbalance13(s)..

$Q(s,"7","12")$ =E= $(V(s,"12") + V(s,"13") + V(s,"14")) * discharge$

;

Equation flowbalance14;

flowbalance14(s)..

$Q(s,"12","13")$ =E= $(V(s,"13") + V(s,"14")) * discharge$

;

Equation flowbalance15;

flowbalance15(s)..

$Q(s,"13","14")$ =E= $V(s,"14") * discharge$

;

Equation flowbalance16;

flowbalance16(s)..

$Q(s,"15","16")$ =E= $(V(s,"16") + V(s,"17") + V(s,"18") + V(s,"19")) * discharge$

;

Equation flowbalance17;

flowbalance17(s)..

$Q(s,"16","17")$ =E= $(V(s,"17") + V(s,"18") + V(s,"19")) * discharge$

;

Equation flowbalance18;

flowbalance18(s)..

$Q(s,"17","18")$ =E= $(V(s,"18") + V(s,"19")) * discharge$

;

Equation flowbalance19;

flowbalance19(s)..

$Q(s,"18","19")$ =E= $V(s,"19") * discharge$

;

Equation flowbalance20;

flowbalance20(s)..

$Q(s,"15","20")$ =E= $(V(s,"20") + V(s,"21") + V(s,"22")) * discharge$

;

Equation flowbalance21;

flowbalance21(s)..

$Q(s,"20","21")$ =E= $(V(s,"21") + V(s,"22")) * discharge$

;

Equation flowbalance22;

flowbalance22(s)..

$Q(s,"21","22")$ =E= $V(s,"22") * discharge$

;

Equation flowbalance23;

flowbalance23(s)..

$Q(s,"23","24")$ =E= $(V(s,"24")+V(s,"25")+V(s,"26")+V(s,"27"))*discharge$

;

Equation flowbalance24;

flowbalance24(s)..

$Q(s,"24","25")$ =E= $(V(s,"25")+V(s,"26")+V(s,"27"))*discharge$

;

Equation flowbalance25;

flowbalance25(s)..

$Q(s,"25","26")$ =E= $(V(s,"26")+V(s,"27"))*discharge$

;

Equation flowbalance26;

flowbalance26(s)..

$Q(s,"26","27")$=E= $(V(s,"27"))*discharge$

;

Equation flowbalance27;

flowbalance27(s)..

$Q(s,"23","28")$ =E= $(V(s,"28")+V(s,"29")+V(s,"30"))*discharge$

;

Equation flowbalance28;

flowbalance28(s)..

$Q(s,"28","29")$ =E= $(V(s,"29")+V(s,"30"))*discharge$

;

Equation flowbalance29;

flowbalance29(s)..

$Q(s,"29","30")$ =E= $V(s,"30")$*$discharge$

;

```
******************************************************************************
* Other constraints and model equations
******************************************************************************
Equation Node-Valves;
Node-Valves(ij(i,j))$((ord(j) ne 2)and (ord(j) ne 7) and (ord(j) ne 15) and (ord(j) ne
23))..
SUM(k, NODE(ij,k)) =E= SUM(s, V(s,j))
;
Equation IN_NODE assure that only one pipeline type k can be chosen;
IN_NODE(ij(i,j), s, k)..
SUM(d$(ijd(ij,d)), IN(ij,s,d,k)) =L= NODE(ij,k);
******************************************************************************
* Variable bounds
******************************************************************************
Loop(ib, P.lo(s, ib) = 40);
P.lo(s, "1") = 40 ;
P.lo(s, "2") = 40 ;
P.lo(s, "7") = 40 ;
P.lo(s, "15") = 40 ;
P.lo(s, "23") = 40 ;
Loop(ib, P.up(s, ib) = 50);
P.up(s, "1") = 70 ;
P.up(s, "2") = 60 ;
P.up(s, "7") = 60 ;
P.up(s, "15") = 55 ;
P.up(s, "23") = 50 ;
******************************************************************************
```

```
Q.up(s, "1","2") = 10*discharge ;
Q.up(s, "2","7") = 10*discharge ;
Q.up(s, "7","15") = 10*discharge ;
Q.up(s, "15","23") = 7*discharge ;
Q.up(s, "2","3") = 4*discharge ;
Q.up(s, "7","8") = 4*discharge ;
Q.up(s, "15","16") = 4*discharge ;
Q.up(s, "23","24") = 4*discharge ;
Q.up(s, "3","4") = 3*discharge ;
Q.up(s, "8","9") = 3*discharge ;
Q.up(s, "16","17") = 3*discharge ;
Q.up(s, "24","25") = 3*discharge ;
Q.up(s, "4","5") = 2*discharge ;
Q.up(s, "9","10") = 2*discharge ;
Q.up(s, "17","18") = 2*discharge ;
Q.up(s, "25","26") = 2*discharge ;
Q.up(s, "5","6") = discharge ;
Q.up(s, "10","11") = discharge ;
Q.up(s, "18","19") = discharge ;
Q.up(s, "26","27") = discharge ;
Q.up(s, "7","12") = 3*discharge ;
Q.up(s, "15","20") = 3*discharge ;
Q.up(s, "23","28") = 3*discharge ;
Q.up(s, "12","13") = 2*discharge ;
Q.up(s, "20","21") = 2*discharge ;
Q.up(s, "28","29") = 2*discharge ;
Q.up(s, "13","14") = discharge ;
```

```
Q.up(s, "21","22") = discharge ;
Q.up(s, "29","30") = discharge ;
******************************************************************************
File OPT Cplex option file / cplex.OPT / ;
Put OPT ;
Put
'nodefileind 2'/
'workmem 2048'/
'threads 0'/;
Putclose OPT ;
ModelModel kalabsha /all/ ;
kalabsha.optfile = 1 ;
Option limrow = 300000 ;
Option reslim = 100000 ;
Option SysOut = On ;
Option mip=CPLEX;
Solve kalabsha maximizing NPW using mip ;
******************************************************************************
* OUTPUT
******************************************************************************
File RESULTS_q/RESULTS_q.csv/ ;
RESULTS_q.pc=5 ;
Put RESULTS_Q
Loop(ij(i,j), put Q.l("1",ij):<>:0,Q.l("2",ij):<>:0, Q.l("3",ij):<>:0, i.tl, j.tl /);
PutcloseRESULTS_q
ExecuteExecute '=shellexecute RESULTS_q.csv'
;
```

```
*****************************************************************************
File RESULTS_V/RESULTS_V.csv/ ;
RESULTS_V.pc=5 ;
Put RESULTS_V
Loop(ib, put V.l("1",ib):<>:0, V.l("2", ib):<>:0, V.l("3",ib):<>:0, ib.tl /)
;
Putclose RESULTS_V
Execute '=shellexecute RESULTS_V.csv'
;
*****************************************************************************
File RESULTS_P/RESULTS_P.csv/ ;
RESULTS_P.pc=5 ;
Put RESULTS_P
LoopLoop(ib, put P.l("1",ib):<>:1,P.l("2",ib):<>:1,P.l("3",ib):<>:1, ib.tl /)
Put / P.l("1","1"):<>:1,P.l("2","1"):<>:1, P.l("3","1"):<>:1,"1"
Put / P.l("1","2"):<>:1,P.l("2","2"):<>:1, P.l("3","2"):<>:1,"2"
Put / P.l("1","7"):<>:1,P.l("2","7"):<>:1, P.l("3","7"):<>:1,"7"
Put / P.l("1","15"):<>:1,P.l("2","15"):<>:1, P.l("3","15"):<>:1,"15"
Put / P.l("1","23"):<>:1,P.l("2","23"):<>:1, P.l("3","23"):<>:1 ,"23"
Putclose RESULTS_P
Execute '=shellexecute RESULTS_P.csv'
;
*****************************************************************************
File RESULTS_PIPE/RESULTS_PIPE.csv/ ;
RESULTS_PIPE.pc=5 ;
Put RESULTS_PIPE
Parameter PIPE_yn;
```

```
PIPE_yn(ij,k) = 100*diam(k)*NODE.l(ij,k);
Loop((ij(i,j)), put SUM(k,PIPE_yn(ij,k)):<>:0 , i.tl, j.tl/);
PutclosePutclose RESULTS_PIPE
*******************************************************************************
File RESULTS_NPW/RESULTS_NPW.csv/ ;
RESULTS_NPW.pc=5 ;
put RESULTS_NPW
Parameter NPV;
NPV = NPW.l
Parameter PW_NBN;
PW_NBN = SUM(ij, SUM(k,return(ij)*NODE.l(ij,k)))*PWef ;
Parameter PW_EC;
PW_EC = EC.l*PWed ;
Parameter d_OandM_costs;
d_OandM_costs = SUM(ij, SUM(k, pprice(k)*arc(ij)*NODE.l(ij,k)))*(0.005)*PW
+ 287.6*POWER.l("1")*0.02*PW ;
Parameter d_depr;
d_depr = SUM(ij, SUM(k, pprice(k)*arc(ij)*NODE.l(ij,k)))*(1/n)*PW
+ 287.6*POWER.l("1")*(1/n)*PW ;
********
Parameter pipelinecosts;
pipelinecosts = SUM(ij, SUM(k, pprice(k)*arc(ij)*NODE.l(ij,k)))
Parameter pumpcost;
pumpcost = 287.6*POWER.l("1") ;
Parameter costn;
costn = SUM(ij, costnode(ij)*SUM(k, NODE.l(ij,k))) ;
***********
```

```
Parameter OandM_costs;
OandM_costs = SUM(ij, SUM(k, pprice(k)*arc(ij)*NODE.l(ij,k)))*(0.005) +
287.6*POWER.l("1")*0.02 ;
Parameter depr;
depr = SUM(ij, SUM(k, pprice(k)*arc(ij)*NODE.l(ij,k)))*(1/n)+
287.6*POWER.l("1")*(1/n) ;
************
ParameterTpower;
Tpower = SUM(s, POWER.l(s)) ;
******************************************************************************
Parameter Ic number of irrigation cycles per year (units);
Ic= (1/6 + 1/10)*140
Parameter Sr irrigated area (ha) multiplied by 2 for two seasons;
Sr = SUM(ij(i,j)$((ord(j) ne 2)and (ord(j) ne 7) and (ord(j) ne 15) and (ord(j) ne 23)),
SUM(k, NODE.l(ij,k)))*20*2
Parameter Vt total annual volume of irrigation water supply (m3);
Vt = SUM(s, Q.l(s,"1","2"))*3.6*19.85*Ic
Parameter VtSr total annual volume of irrigation water supply per unit irrigated area
(m3 per ha);
VtSr = Vt/Sr
Parameter Thrs total hours of operation per year (h);
Thrs = 3*19.85*(1/6 +1/10)*140
Parameter AAP average hydraulic power (kW) ;
AAP = (1/3)*SUM(s, POWER.l(s))
parameter ACE average annual consumed energy (kWh);
ACE = Thrs*AAP
Parameter AAPSr average hydraulic power per unit irrigated area (kW per ha);
```

AAPSr = AAP/Sr

Parameter ACESr annual consumed energy per unit irrigated area (kWh per ha);

ACESr = ACE/Sr

Parameter ACEVt annual consumed energy per unit irrigation supply (kWh per m3);

ACEVt = ACE/Vt

Parameter ACD annual equivalent consumed diesel (L);

ACD = ACE*0.23

Parameter DCegSr diesel costs per irrigated area (EGP per ha) ;

DCegSr = (ACD/Sr)*5.88

Parameter DCeuSr diesel costs per irrigated area (EUR per ha) ;

DCeuSr = (ACD/Sr)*5.88/8

Parameter ABeuSr annual benefits per irrigated area (EUR per ha);

ABeuSr = (1/Sr)*(SUM(ij(i,j)$((ord(j) ne 2)and (ord(j) ne 7) and (ord(j) ne 15) and (ord(j) ne 23)), SUM(k, NODE.l(ij,k))* return(ij))/8)

Parameter DCegVt diesel costs per unit irrigation supply (EGP per m3);

DCegVt = (ACD/Vt)*5.88

Parameter DCeuVt diesel costs per unit irrigation supply (EUR per m3);

DCeuVt = (ACD/Vt)*5.88/8

ParameterABeuVt annual benefits per unit irrigation supply (EUR per m3);

ABeuVt = (1/Vt)*(SUM(ij(i,j)$((ord(j) ne 2)and (ord(j) ne 7) and (ord(j) ne 15) and (ord(j) ne 23)), SUM(k, NODE.l(ij,k))* return(ij))/8)

****

Put "", "net present value", NPV /

Put "", "discounted node's net benefits", PW_NBN /

Put "", "discounted energy costs", PW_EC /

Put "", "discounted OM costs", d_OandM_costs /

Put "", "discounted depreciation costs", d_depr /

```
Put "", " " /
Put "", "Investment Costs" /
Put "", "pipeline network", pipelinecosts /
Put "", "pump system", pumpcost/
Put "", "farm's irrigation systems", costn /
***********
Put "", "operation and maintenance costs per year", OandM_costs /
Put "", "depreciation costs per year", depr /
********************************************************************************************
Put "****************************************************************************" /
Put " " /
Put "Sr", "irrigated area (ha)", Sr /
Put "VtSr", "total annual volume of irrigation water supply per unit irrigated area (m3 per ha)", VtSr:<>:3 /
Put "Vt", "total annual volume of irrigation water supply (m3)" , Vt:<>:3 /
Put "Ic", "number of irrigation cycles per year (units)" , Ic /
Put "Thrs", "total hours of operation per year (h)", Thrs:<>:3 /
Put "AAP", "average measured absorbed power (kW)", AAP:<>:3 /
Put "ACE", "annual consumed active energy (kWh)", ACE:<>:3 /
Put "AAPSr", "average measured absorbed power per unit irrigated area (kW per ha)", AAPSr:<>:3 /
Put "ACESr", "annual consumed active energy per unit irrigated area (kWh per ha)", ACESr:<>:3 /
Put "ACEVt", "annual consumed active energy per unit irrigation supply (kWh per m3)", ACEVt:<>:3 /
Put "ACD", "annual equivalent consumed diesel (L)", ACD:<>:3 /
```

```
Put "DCegSr", "diesel costs per irrigated area (EGP per ha)", DCegSr:<>:3 /
Put "DCeuSr", "diesel costs per irrigated area (EUR per ha)", DCeuSr:<>:3 /
Put "DCegVt", "diesel costs per unit irrigation supply (EGP per m3)", DCegVt:<>:3
/
Put "DCeuVt", "diesel costs per unit irrigation supply (EUR per m3)", DCeuVt:<>:3
/
Put "ABeuVt", "annual benefits per unit irrigation supply (EUR per m3)",ABeuVt:<>:3
/
Put "ABeuSr", "annual benefits per irrigated area (EUR per ha)", ABeuSr:<>:3 ;
********************************************************************************
Execute '=shellexecute RESULTS_DISSERTATION.xls'
```

Appendix E

A PYTHON soft-linkage between GAMS and ArcGIS

E.1 Introduction

In this dissertation the results of the optimization model are presented in ArcMap. For this purpose a geoprocessing Python script was written creating a softlinkage between the output of the optimization program written in GAMS and the ArcMap component of ArcGIS. The Python scrip is applied to the ArcMap model of the Kalabsha settlement. The ArcMap model is composed of an Digital Elevation Model (DEM), which allows the display of the countor curves of the region. In ArcMap shape files are contained, which represent each of the different pipelines of the system and also the different nodes. The main task of the Python script is to manage the different pipelines and node shape files according to the solutions given by the GAMS optimization model. This means, if the solution indicates that node 30, and pipeline $(i, j) = (29, 30)$, are not in the optimal solution, the Python script will disable the corresponding shape files, i.e. the network structure appearing in ArcMap will exclude this pipeline and node.

E.2 The python script

A python script is commonly build on several distinct blocks. In the *first block*, we import to Python several modules or packages, that are necessary for the needed processes, when running the script. In this block and for our purpose, the modules imported are: *arcpy, sys, traceback, datetime*, and *xlrd.* The *arcpy* module guarantees access to the all the geoprocessing functions of ArcGIS. The *sys* module accesses Python system functions, e.g. for defining variables for user input. The *traceback* module is used for bug tracking and error handling. The datetime module manages the indications of datetimes, and the *xlrd* is the module or package that allows the reading data and formatting information from Excel files. This package is at the center of the softlinkage, because the results coming from the optimization model in GAMS are first dropped into Excel. The Python script reads the results dropped in Excel and manages accordingly the geospatial information to be displayed in ArcMap.

```
#1. IMPORT MODULES
import arcpy, sys, traceback, datetime, xlrd
arcpy.env.workspace = 'C:\\GIS\\Egypt\\Kalabsha\\'
from xlrd import open_workbook, cellname # xlrd.sheet.Sheet, where Sheet
sind attributes (e.g. .name, .nrows, .ncols)
from arcpy.mapping import * # Eliminates the need to write arcpy.mapping
# before each mapping module related class, function or method
# Create date
CUR_DATE = datetime.date.today().strftime('%m.%d.%Y')
```

The second and third blocks are named try: and except: blocks. In blocks try: is where the main code lines are going to be grouped. The *except* block is the final block reserved for error handling.

try:

In this block the excel workbook will be opened and local variables defined

```
bookARCNODE = open_workbook(arcnodesTable)
sheetARCS = bookARCNODE.sheet_by_index(0)
sheetNODES = bookARCNODE.sheet_by_index(1)
bookVALPIPE = open_workbook(VALPIPEbook)
sheetPIPES = bookVALPIPE.sheet_by_index(4)
sheetVALVES = bookVALPIPE.sheet_by_index(3)
# Read from Excel
col_index_0 = 0
col_index_1 = 1
col_index_2 = 2
col_index_3 = 3
```

A loop starts through all the layers (shape files) available in the list of layers of the Kalabsha ArcMap model.

```
for TOCLayer in ListLayers(mxd):
for row_index in range(sheetARCS.nrows):
arcName = sheetARCS.cell(row_index, col_index_0).value
PIPE = sheetPIPES.cell(row_index, col_index_0).value
```

The conditions are not set-up for handling the pipelines and nodes in ArcMap. The different layers (pipelines and nodes) are made visible or invisible according to the results recorded in the Excel book and sheets.

```
if TOCLayer.name == arcName and PIPE >= 1:
TOCLayer.visible = True # turn layer on
```

```
print TOCLayer.name + ' is visible'
elif TOCLayer.name == arcName and PIPE <= 0:
TOCLayer.visible = False # turn layer off
print TOCLayer.name + ' is NOT visible'
for row_index in range(sheetARCS.nrows):
nodesNAME = sheetARCS.cell(row_index, col_index_1).value
if TOCLayer.name == nodesNAME + '_2' and PIPE<= 0:
TOCLayer.visible = False # turn layer off
print TOCLayer.name + ' is NOT visible'
for row_index in range(sheetNODES.nrows):
nodesNAME = sheetNODES.cell(row_index, col_index_0).value
ONOFF1 = sheetVALVES.cell(row_index, col_index_0).value
ONOFF2 = sheetVALVES.cell(row_index, col_index_1).value
ONOFF3 = sheetVALVES.cell(row_index, col_index_2).value
if TOCLayer.name == nodesNAME +'_ON1' and ONOFF1 >=1:
TOCLayer.visible = True # turn layer on
print TOCLayer.name + ' is ON'
elif TOCLayer.name == nodesNAME + '_ON1' and ONOFF1 <=0:
TOCLayer.visible = False # turn layer off
print TOCLayer.name + ' is OFF'
if TOCLayer.name == nodesNAME +'_ON2' and ONOFF2 >=1:
TOCLayer.visible = True # turn layer on
print TOCLayer.name + ' is ON'
elif TOCLayer.name == nodesNAME + '_ON2' and ONOFF2 <=0:
TOCLayer.visible = False # turn layer off
print TOCLayer.name + ' is OFF'
if TOCLayer.name == nodesNAME +'_ON3' and ONOFF3 >=1:
```

```
TOCLayer.visible = True # turn layer on
print TOCLayer.name + ' is ON'
elif TOCLayer.name == nodesNAME + '_ON3' and ONOFF3 <=0:
TOCLayer.visible = False # turn layer off
print TOCLayer.name + ' is OFF'
mxd.saveACopy(r"C:\GIS\Egypt\Kalabsha\Kalabsha4.mxd")
#mxd.save()
```

The results are saved. The next block *except:* contains code for error tracking only and will not be showed here.

Appendix F

Soil analysis

Figure F-1: Soil analysis performed by the MALR in the framework of the BMBF OWARA research project.

Ministry of Agriculture
and Land Reclamation
World Food Programme
Proj.: 3214/3
velopment and Settlement
Center for Bio-Organic
Agricultural Services

وزارة الزراعة وإستصلاح الأراضى
برنامج الأغذية العالمى
مشروع ٣/٣٢١٤
تنمية وتوطين بحيرة السد العالى
مركز خدمات الزراعة الحيوية
أسوان

تحريراً فى / /

تقرير عن التحاليل الكيماويه والطبيعية لعينات التربة المأخوذة من الأراضي الزراعية بمنطقة كلابشه

****

١. تم تحليل عدد ٩٠ عينة تربة لعدد ٣٠ قطاع بكل قطاع ٣ أعماق (سطحيه – تحت السطحية – عميقة
وجد من التحليل الطبيعي **physical analysis** لهذه العينات زيادة نسبة الرمل sand في معظم القطا
حيث تراوحت النسبة من ٧٧.٤ – ٩٨.٤% وقد تباينت نسبة السلت **silt** في هذه العينات من منخفض
متوسطة من ١.٦ – ٢٠.٩ % بينما انخفضت نسبة الطين **clay** جداً وتراوحت النسب في العينات من ص
٧.٦ % وكان تصنيف هذه العينات من حيث **soil texture** من رمليه sandy إلى رمليه س
. sandy loam

Soil porosity ??

٢) وجد أن معدل تشبع التربة بمياه الري من متوسط إلى مرتفع حيث تراوحت قيم S.P. من 20.3 إلى 35.3

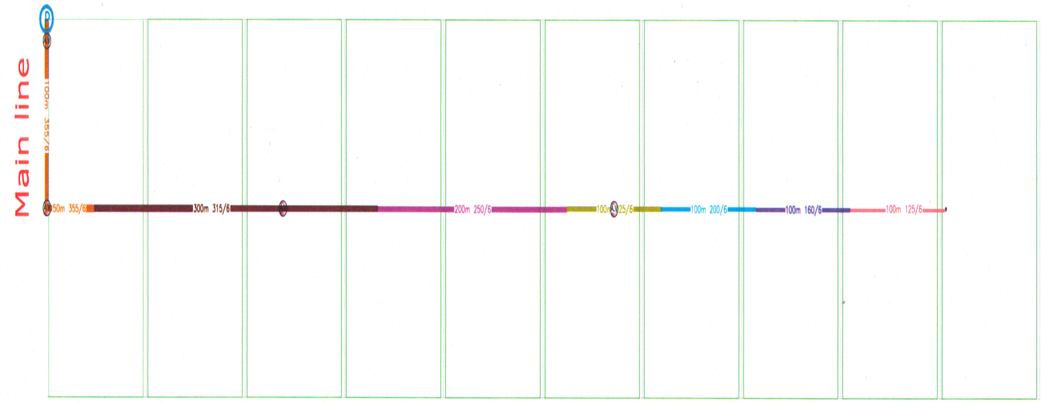
Appendix G

Irrigation system's design at the hydrant level

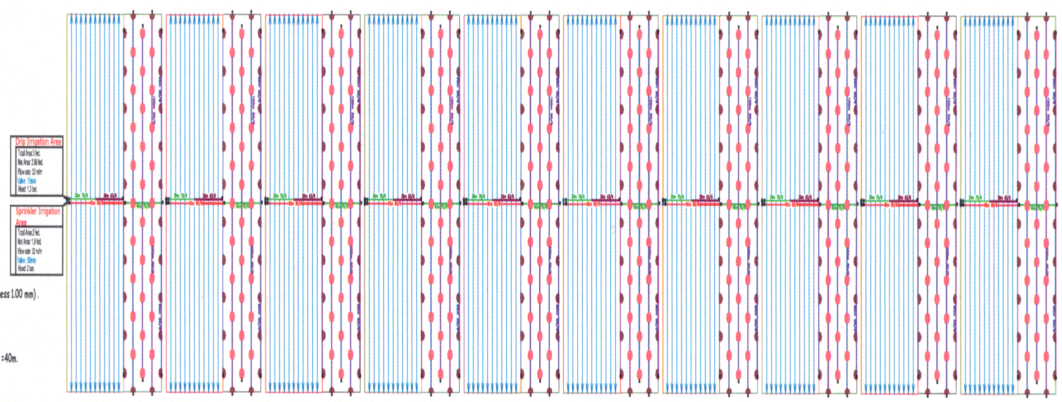
A49

Legend:

PIPE ø355	
PIPE ø315	
PIPE ø250	
PIPE ø225	
PIPE ø200	
PIPE ø160	
PIPE ø125	
PIPE ø110	
PIPE ø90	
PIPE ø75	
PIPE ø63	
PIPE ø50	
LATERAL	
PUMPING CENTER	P
VALVE	
AIR VALVE 1"	A.V
AIR VALVE 2"	A.V
End flushing	



Irrigation Data		Crop
Irrigation system		Drip Irrigation
Emitter Type		DripNet 16mm.
Emitter Spacing	m	0.6
No. of drip lines per row		1
Distance between rows	m	1
Emitter Discharge	l/h	1.6
Duration of one operation	hr	12
Daily duration		12



Design Notes

1) The area is designed for one shift.
2) Drip laterals for citrus = DripNET 1.6/.6/16 (wall thickness 1.00 mm).
3) Submains - subsurface P.V.C. pipes Class 6.
4) Main-lines - subsurface P.V.C. pipes Class 6.
5) Pump / system requirements: Flow=650 m3/h , Pressure =40m.

Appendix H

Irrigation system's costs at the hydrant level

Supplies for each farmer

Item	Made by	Quantity	Unit Price	Total Price
	P.V.C. Pipeline			
P.V.C. pipeline Ø 90 mm - 6 bar	Misr Elnur	78	8.66	675.48
P.V.C. pipeline Ø 90 mm - 6 bar	Misr Elnur	90	6	540.00
P.V.C. pipeline Ø 90 mm - 6 bar	Misr Elnur	48	4.31	206.88
			Total Price	**1,422.36**

Item	Made by	Quantity	Unit Price	Total Price
Reducer 75/90	COMER	1	12.36	12.36
Reducer 63/75	COMER	1	8.25	8.25
Clamp Saddle 75/ 1.5"	plassim	5	17.75	88.75
Top of the line 75 /63 /1.5" P.V.C	COMER	5	6.85	34.25
P.V.C Tee 50 mm	COMER	5	6.3	31.50
Clamp Saddle 50/ 1/2"	plassim	52	25	1,300.00
Riser U.P.V.C 1/2" with 2 m length	Misr Elnur	52	25	1,300.00
			Total Price	**2,775.11**

Item	Made by	Quantity	Unit Price	Total Price
Elbow 75 mm / 45°	COMER	2	17.2	34.40
Elbow 63 mm / 45°	COMER	2	8.4	16.80
Elbow 50 mm / 45°	COMER	20	5.4	108.00
Valve 75 mm with one Blakcor	COMER	1	181.5	181.50
Valve 63 mm with one Blakcor	COMER	1	85.8	85.80
Valve 50 mm with one Blakcor	COMER	10	61.4	614.00
			Total Price	**1,040.50**

Item	Made by	Quantity	Unit Price	Total Price
	Tasks valves			
Blakcoh valve 90 mm P.V.C	COMER	1	246.8	246.80
Blakcoh valve 75 mm P.V.C	COMER	1	186.5	186.50
			Total Price	**433.30**

Item	Made by	Quantity	Unit Price	Total Price
Air relase valve 1"	ARI	3	245	735.00
Saddle 90 mm / 1"	plassim	1	20.1	20.10
Saddle 75 mm / 1"	plassim	2	17.75	35.50
Riser U.P.V.C 1" with 2 m length	مصر النور	3	12	36.00
			Total Price	**826.60**

Item	Made by	Quantity	Unit Price	Total Price
Hose - DripNET 1.6 l/h /0.6m	NETAFIM	12500	1.35	16,875.00
P.E Solid Hose 16 mm	NETAFIM	400	1.35	540.00
P.E Gromet	NETAFIM	200	1.85	370.00
End Cap P.E	NETAFIM	200	0.25	50.00
P.E Connected joint 17 /17	NETAFIM	200	0.75	150.00
Sprinkler 1/2" full cycle	Rain Bird	28	27	756.00
Sprinkler 1/2" halfe cycle	Rain Bird	24	25	600.00
			Total Price	**19,341.00**

Total tasks of each individual farm **25,838.87**

Tasks of the main pipeline				
Item	Made by	Quantity	Unit Price	Total Price
P.V.C. pipeline Ø 355 mm - 6 bar	Misr Elnur	228	165.25	37,677.00
P.V.C. pipeline Ø 315 mm - 6 bar	Misr Elnur	330	100.67	33,221.10
P.V.C. pipeline Ø 250 mm - 6 bar	Misr Elnur	228	63.43	14,462.04
P.V.C. pipeline Ø 225 mm - 6 bar	Misr Elnur	120	50.2	6,024.00
P.V.C. pipeline Ø 200 mm - 6 bar	Misr Elnur	120	41.16	4,939.20
P.V.C. pipeline Ø 160 mm - 6 bar	Misr Elnur	120	26.37	3,164.40
P.V.C. pipeline Ø 125 mm - 6 bar	Misr Elnur	120	16.33	1,959.60
			Total Price	101,447.34

Item	Made by	Quantity	Unit Price	Total Price
P.V.C Elbow 355 mm	COMER	1	4050.75	4,050.75
P.V.C Reducer 355 /315	COMER	1	877.75	877.75
P.V.C Reducer 315 /250	COMER	1	625.94	625.94
P.V.C Reducer 250 /200	COMER	1	605.63	605.63
P.V.C Reducer 200 /160	COMER	1	88.32	88.32
P.V.C Reducer 160 /125	COMER	1	41.5	41.50
Air relas valve 2"	ARI	4	580	2,320.00
P.V.C Saddle 355 mm / 2"	plassim	2	489.98	979.96
P.V.C Saddle 315 mm / 2"	plassim	1	489.98	489.98
P.V.C Saddle 225 mm / 2"	plassim	1	379.65	379.65
U.P.V.C Riser 2" with 2 m length	Misr Elnur	4	26	104.00
Elbow 125 mm / 45°	COMER	2	65	130.00
P.V.C valve 125 mm with one Blakcoh	COMER	1	555	555.00
P.V.C Tee 355 mm	COMER	1	4607.65	4,607.65
P.V.C Tee 315 mm	COMER	3	2106.25	6,318.75
P.V.C Tee 250 mm	COMER	2	1141	2,282.00
P.V.C Tee 225 mm	COMER	1	559.35	559.35
P.V.C Tee 200 mm	COMER	1	492.55	492.55
P.V.C Tee 125 mm	COMER	1	104.4	104.40
P.V.C Tee 90 mm	COMER	10	33.1	331.00
P.V.C Reducer 355 /90	COMER	1	736.1	736.10
P.V.C Reducer 315 /90	COMER	3	736.1	2,208.30
P.V.C Reducer 250 /90	COMER	2	523.41	1,046.82
P.V.C Reducer 225 /90	COMER	1	181.2	181.20
P.V.C Reducer 200 /90	COMER	1	150.64	150.64
P.V.C Reducer 100 /90	COMER	1	126.75	126.75
P.V.C Reducer 125 /90	COMER	1	53.2	53.20
P.V.C Reducer 90 /75	COMER	10	8.15	81.50
Elbow 90 mm / 90°	COMER	10	31.35	313.50
Elbow 75 mm / 90°	COMER	10	12.55	125.50
		Total Price		30,967.69