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E.T.S INGENIEROS DE CAMINOS, CANALES Y PUERTOS

NEW DESIGN METHODOLOGY FOR BRANCHED
HYDRAULIC NETWORKS THAT ACCOUNTS FOR VARIABLE
WATER DEMAND AND ENERGY ASSESMENT

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Abstract

The design of water supply systems has to consider all costs involved from the moment of its construction until the end of the installation, for it to be effective and sustainable. This implies accounting not only for the building expenses but also for the energy investments of the operation of the facility when pumping is required. A key variable for the calculation of the electrical costs is the value of the pump efficiency. Up to now, the prediction of the pump efficiency was made using conservative values, in the absence of a better estimation. On the other hand, because water demand varies over time depending on numerous factors, such as seasonal irrigation, or population displacements, etc., it is desirable that the design methodology is able to adequately manage this phenomenon.

For these reasons, the aim of this work is to present a new design methodology for water supply systems for branch networks that effectively accounts for energy expenses as well as variable demand patterns. The methodology will be based on the Granados' System, which is a very practical and intuitive gradient based procedure. This thesis intends to carry out an extensive pump analysis with the purpose of defining the expected value of the pump efficiency and in this way simplify Granados' calculation process. At the same time, because Granados' System requires a constant design flow rate, this thesis also pursues to provide a solution by which variable demand distributions can be converted to constant regimes.

For the pump efficiency analysis, 226 commercial pumps were studied in depth, collecting the characteristic features at their optimum operation point. As a result, a strong relationship between the pump efficiency and the flow rate was spotted, which allowed the elaboration of an empiric curve that designers can use to predict the expected value of the pump efficiency, and thus perform a proper energy assessment. This also simplifies calculations in the Granados' System by braking an iterative process for guessing the pump efficiency.

Regarding the variable water demand issue, this work presents a straightforward methodology that, using a constant flow rate, the total cost is equivalent to that of the variable demand flow. Following this logic, the concepts of the Equivalent Flow Rate and the Equivalent Volume are developed. These concepts can be

applied to, not only the Granados' System, but also to any other design methodology that requires a constant flow rate as input.

Combining all of these contributions and reorganising the steps of the procedure, a new design methodology for branch water supply networks is proposed and applied in a case study to fully illustrate its use.

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Introduction

Motivation

If there is something that characterises nowadays society, it is the increase of exigency, in every aspect. Globalisation has brought human kind the possibility of stablishing anywhere, making cities grow as rapid as never before; it has allowed individuals to seasonally move from one location to another, and it has boosted population growth increasing food necessities and therefore agricultural needs associated to it. To those fluctuations of people, changes in our planet, and the increasingly demanding culture, water supply systems play a major role, and we need to be prepared.

Water demand patterns are conditioned by population shifting from places, generally associated to a seasonal phenomenon that can leave the crowdest city as a phantom town, and vice versa. These temporal migrations can leave striking figures such as the case of the Meca, which has a population of 1.5 million habitants, and that over the Hajj peregrination season can host over 2 million additional people.ⁱ

However, drinkable water variations are not the only source of irregularity in the demand patterns. Agricultural purposes represent 87% of the total worldwide water consumption.ⁱⁱ Needless to say, different crops require different irrigation, bringing complexity to the design of water supply systems. Climate change is now accentuating these variations since summer months are heading into dryer conditions, incrementing the need for water in those periods; and raining seasons are becoming shorter and more aggressive, hindering natural irrigation.

With all this in mind, demand variability is a crucial factor that cannot be dismissed from the design process of a water supply system. How to incorporate demand fluctuations into designing procedures will be one of the main pillars of this investigation.

ⁱ See *Deutsche Welle* (2018)

ⁱⁱ See *Fundación Aquae*: www.fundacionaquae.org

Nevertheless, an effective design is the one that also optimises the use of the available resources and funds. This is especially decisive when the design in particular is located in a developing area, with limited capacities. Finding the design solution that is aware all costs derived from the utility is key. Not all costs are diverted from the construction, but in fact operational expenses are essential a successful use of the facility, and they need to be accounted from the beginning. Not considering energy expenditures when pumping systems are required can drive the installation to the ruin.

In September 2015 the United Nations published the 2030 Agenda for Sustainable Development (W. Colglazier (2020))⁸. The agenda intended to eradicate poverty in all forms, by stablishing 17 goals (Sustainable Development Goals SDG) and 169 objectives to foster sustainable development through economic, social and environmental aspects. Goal 6 –Water Availability– intends to achieve universal access to drinkable water and sanitation, and it is very much in relation with Goal 7 –Sustainable Energy– which seeks to guarantee sustainable energy for all, since a big fraction of the costs derived from water transportation is associated with water pumping. For the design of water supply systems it must be found a balance point between construction costs and operation expenses, to therefore achieve a proper water-energy nexus. In order to fulfil these challenges, an efficient design of water supply system is fundamental. Sizing methodologies of water supply systems need to be practical, intuitive for designers, reliable and affordable. This thesis bases its second pillar on these principles, and pursues to contribute to the cause of goal 6- Water Availability-, 7-Sustainable Energy-, by developing goal 9 –Innovative Infrastructure-, to create goal 11 –Sustainable Communities- and fight for goal 13-Climate Action- by carrying out a proper energy evaluation.

With all this in mind, the purpose of this thesis is to propose a design methodology of water supply systems in accordance with the challenges of today's world: Integrating the demand variability into the sizing processes and becoming energetically efficient by considering energy expenses from the conception of the network.

Some of the results presented on this thesis have already been published and validated by the scientific community in the form of two research articles that were recently published in January of 2020 (A. Martín-Candilejo et al. (2020))⁴⁸ and December 2019. (A. Martín-Candilejo et al. (2019))⁴⁷. This thesis has been supported and financed by Carlos Gonzalez Cruz Foundation, to which this research team is deeply grateful.

Aim of the research

Professor Alfredo Granados is the author of the Granados' System^{28, 46}: It is a well-recognised methodology for pipe sizing of branched networks. It is a practical procedure that accounts for the energy costs of the utility when pumping is required. It has been widely used for its intuitive logic, and it belongs to the Gradient Based procedures category. Nevertheless, Granados' System requires a constant design flow rate for its application (as well as many other design methods), and depends on an iterative process to determine the expected pump efficiency, which substantially affects the energy cost. This investigation intends to adapt Granados' System to a variable flow rate demand pattern, and provide an accurate definition of the pump efficiency in order to simplify the procedure. Then, the designing steps will be reorganised incorporating the new contributions, to conform a new design methodology that has a wider application scope but also benefits from the practicality of the original Granados' System.

Therefore, this thesis has three main goals:

1. Variable Demand Adaptation: Since the logic of the Granados' System requires a constant design flow rate, the aim is to transform a variable flow rate regime to an equivalent constant demand distribution. For this, two new concepts are developed, for two different techniques:
 - ▶ The Equivalent Flow Rate.
 - ▶ The Equivalent Volume.
2. Pump Efficiency Definition: A wide sample of commercial pumps will be studied in terms of their pump efficiency and its relationship with other design variables. From this in depth analysis, a definition curve will be obtained.
3. Reorganisation of the methodology and application of it: The above contributions are introduced in the Granados' System. Since the definition of the pump efficiency helps avoiding an iterative process from the original method, the steps are reorganised into a new methodology. To illustrate its validity, the method is then applied into a complete full case study of the design of a branch network.

Figure I.1. Goals of the thesis

Altogether, the objectives then are:

- ▶ Find an equivalence between a variable flow rate demand and a comparable constant regime.
- ▶ Definition of the Equivalent Flow Rate.
- ▶ Definition of the Equivalent Volume.
- ▶ Determine if the pump efficiency is related to any other design variable.
- ▶ Give a precise estimation of the pump efficiency.
- ▶ Incorporate all steps into a one methodology.
- ▶ Simplify calculations.
- ▶ Apply the methodology in a complete case study.
- ▶ Verify the validity of the new methodology.

It should be kept in mind that this is not only useful for the purposes of Granados' Method, but also for any of the other design procedures that work under the same circumstances: Needing a constant flow rate as an input and a pre-set value of the pump efficiency.

Structure of the thesis

This thesis is divided in six chapters:

The first chapter is the introductory and it will cover the State of the Art, as well as the present preamble. The current state of knowledge will discuss the various design methodologies, the objectives that they consider, programming techniques, and what is the current trend when designing a water supply system.

Moving onto the second chapter, the Methodology is presented. The methodology starts with a brief summary of the Granados' System, and the explanation of its limitations. Then, Goal 1: Variable Demand Adaptation, is addressed: The concepts of the Equivalent Flow Rate and Equivalent Volume are conceptually explained and mathematically deduced. There is a second part in the methodology, and that is the description of the pump efficiency analysis sample and procedure. With this, the chapter is concluded.

The third chapter refers to the Results and Discussion, and it can be divided into the following sections: Pump Efficiency Analysis results complemented with the discussion of these; followed by the Assembling of the full procedure, where all steps are reorganised into the new methodology and discussed.

On the fourth chapter, the Case Study is presented. It begins with a description of the network, followed by a presentation of the variable demand regime. Then calculations are carried out, and the optimum design is obtained. To validate the mathematical deductions of the methodology, a comparison between the variable demand and the equivalent constant regime is shown. This chapter is accompanied by an Annex at the very end of the document, that gathers all calculations tables, so that the reading is not disturbed along the chapter.

The fifth chapter relates to the Conclusions, bringing in a summary of the thesis, key results, implications, limitations and future advancement are underlined.

At the very end of the thesis, in chapter six, it can be found the Bibliography document and the Annex of the calculations that was previously announced in the explanation of the case study.

I. Current State of Knowledge

Under the threat of climate change, becoming energy efficient is now a crucial need for our society. Therefore, an adequate energy evaluation is an essential factor to consider in the development of new policies that ensure sustainable designs of water supply systems (R. Sanchis et al. (2019))⁷¹. The design phase of a water distribution system demands considering not only the construction phase but also the operation of the facility throughout its entire useful life. Since the areas with the greatest need for action are underdeveloped areas with scarce economic resources (S. García de Jalón et al. (2014))¹⁷ design strategies must take into account not only construction costs, but also the operation of the system, especially when pumping is required. Energy expenditure represents a huge fraction of the final cost, and should not be excluded from the calculations. There are many design procedures that incorporate this aspect into their analysis. Mala-Jetmarova (2017)⁴¹ does a remarkable job summarising what each author considered in each procedure. After her review of the state of knowledge, it seems that the designers of water distribution systems increasingly emphasize the importance of including evaluations of the operational aspects. Also, she establishes that the optimization procedures can be classified according to the optimization objectives (single objective, multiple objective, etc.) or the calculation method (stochastic, heuristic, etc.).

1.1. Design Objectives

Figure 1.1. summarises the most common objectives in the optimization of water supply systems. Design procedures can be classified in single objective (this is the traditional approach, and it continues to be used nowadays very commonly) and multi objectives. Multi objectives methodologies started being developed in 1990s and there are becoming more popular nowadays. However, these methodologies are still less common than single objective procedures. The most common objective is the "Lowest cost", and it may be combined with other goals such as maximum benefit, or pressure requirements, greenhouse emissions, etc. Sometimes, the objectives combination can be contradictory, because, what means least cost solution, generally represents the highest environmental cost, and vice versa.

Regarding the objectives, methods of one objective (being the "lowest cost" generally) represent 70% of the known methods. However, many design proposals used to consider construction costs as the only cost to account for, leading to an incomplete approach to water management. But striking figures highly encourage against this approach nowadays. In fact, in 1994, according to J.H Kim (1994)³⁵ it was reported that "for some public water supply companies, 90% of the total budget is for energy required to pump".

Figure 1.1. Design objectives in methodologies for the design of water supply systems.

Nevertheless, from the 1975s onward, some designers had already begun to integrate both construction cost and operating cost into the design methodology, like J. Gessler et al. (1985)²⁰ whose construction cost formulation presents excellent detailing, including, for example, excavation costs; or E. Alperovits et al. (1977)¹ whose pumping costs are calculated from an empirical approach

depending on the power required, providing some remarkable results. Some newer designers that include construction and operating costs are A. Ostfeld et al. (2005)⁵⁵ who use EPANET for design; or M. Perez Sanchez et al. (2017)⁵⁸; D. Kang and K. Lansley (2011)³²; X. Jin et al. (2008)³⁰ or H. M. Samani (2006)⁷⁰ who perform a lowest cost evaluation, using hybrid programming; and many others. H.M.Ramos et al. (2020)⁶⁵ analyse the importance of accounting for energy costs by developing a smart water grids. In their work they show the advantages and saving that can be obtained from developing control measures (monitoring water losses and pressure control).

Operating costs are often frequently inserted in optimization algorithms in a single economic function that would also include pipeline construction costs. Classic studies like J. H. Kim (1994)³⁵ presented this approach, and it is still nowadays the most classical way to evaluate energy costs in the sizing process. This is the case of D. Kang. (2012)³³; R. Dziedzic, (2015)¹³; H. M. Samani (2006)⁷⁰; A. Costa (2000)⁹; L. Vamvakieridou-Lyroudia et al. (2005)⁸⁹; M. Spiliotis (2007)⁸²; or X. Jin et al. (2008)³⁰. However, a different perspective is to consider construction and operating costs as separate targets, as E. Roshani (2013)⁶⁹ does. However, logically, in their evaluation they confirmed that this approach yields very different design solutions: the optimal design for construction costs means the highest operating investment and vice versa. This perspective makes it difficult to find the optimum for the general facility, and therefore it is more advisable to work with a single cost function to optimise.

Ideally, water supply systems designing processes should account for as many costs to minimize as possible, in order to have a more realistic picture of the infrastructure performance. Filion et al (2007)¹⁵ focusses on accounting the costs derived from possible failure during low and high pressure water drive. They account for fire circumstances and pipe breaking in their stochastic approach. When it comes to accounting for maintenance costs, it is essential to know the probability in which these events may occur. A. Martinez-Codina (2015)⁵⁰ proposes a very detailed statistical analysis methodology for pipe breaks in water distribution networks, which analyses the relationship between breaks and water pressure and proposes the implementation of pressure management that reduces the number of breaks that occur in these networks. This same approach is used in A. Martinez-Codina et al. (2016)⁵² as well as in A. Martinez-Codina et al. (2015)⁵⁰, where a methodology is developed to identify the range of operation of maximum pressure that is most likely to reduce pipe breaks.

In order to achieve a more accurate evaluation of all the costs involved, multi-objective algorithms have begun to incorporate maintenance costs into the

optimization function, like L. Perelman (2007)⁵⁷ and L. Perelman et al. (2008)⁵⁸; replacement, like the case of W. Wu et al. (2008)⁹⁷; W. Wu et al. (2009)⁹⁵; W. Wu et al. (2009)⁹⁶; greenhouse gas emissions, as W. Wu et al. (2011)⁹⁸; W. Wu et al. (2011)⁹⁹; S. C. Stokes et al. (2015)⁸³; or S. C. Stokes et al. (2015)⁸⁴; carbon foot print, like M. Perez-Sanchez (2017)⁶⁰; pressure deficit, like M. Cisty et al. (2016)⁷; or pressure excess like F. Zheng et al. (2015)¹⁰¹; or water quality, like M. Shokoohi et al. (2017)⁷⁶ and A. López Jiménez (2003)³⁹, among others various targets. Methods that use multiple objectives provide a fuller review of all costs, but often require complex programming, resulting in computational costs and difficulties to use. P. M. Reed et al. (2013)⁶⁷ explained in depth the state of the art of multi-objective techniques, their advancements, programming methods, and their drawbacks. These multi objective proposals represent 30% of the known methodologies according to Mala-Jetmarova (2017)⁴¹.

1.2. Programming techniques

As a general rule, optimization techniques are classified between deterministic methods and stochastic optimization. Deterministic methods have been the traditional approach, but in the 1990s, stochastic methods started developing (initially using Genetic algorithms), and they quickly took over the sector, becoming the most popular approach. Since the past decade, hybrid methods (consisting of a mixture of stochastic and deterministic algorithms) have been used, revelling a combination with very good results. **Figure 1.2.** shows schematically the division of the most common programing techniques. Stochastic optimization methods consists on using and generating random variables. Partly random input data are introduced, and even sometimes, the objective functions and constrains are randomly defined. The most common stochastic techniques are evolutionary algorithms, and more specifically, genetic algorithms. These are inspired by the process of natural selection, and they rely on biologically inspired operators such as mutation, crossover and selection. Deterministic techniques are called like so because they pursue to determine the rigorous optimal solution, with a theoretical guarantee behind.

Most of the efforts and advancements made in modern models have to do with computational processes. In terms of the calculation techniques, evolutionary programming techniques, such as genetic algorithms, have been widely used in the recent years for optimizing distribution systems of water, as A. Marchi et al. (2012)⁴²; S.M. Kadu (2008)³¹ or M. Van Dijk et al. (2008)⁹⁰.

Babayan et al. (2005)⁴ test out the robustness of a network using a different approach other than the very well-known Monte Carlo simulation. Monte Carlo simulation is a stochastic model, but in their work, they convert it into a deterministic model, in such a way they are able to now use Genetic Algorithms. This is a practical way to limit the calculation needs of stochastic methodologies. In this way, they perform a successful robustness analysis when demand varies, and therefore, nodal pressure varies too.

Figure 1.2. Optimization techniques classification.

Todini (2000)⁸⁵ optimises two objective functions: cost and resilience, using a cost gradient methodology. In their work, the diameters are initially selected and then reduced until they touch maximum velocity limits or resilience constrains. In the same line, Gomes et al. (2000)²² propose an iterative dynamic calculation process in their model, as well as a hydraulic simulation to verify their calculations. For this, they utilise EPANET2 as software interface. Their model, called Lenhsnet accounts for the network capital cost, plus the capitalised cost of pumping energy. Another example of the use of EPANET is the case of Reca and Martinez (2006)⁶⁶,

which in this case, they use a genetic algorithm for Pipe Optimization. The model required very little adjustments to the particular case study, and it was tested to find the least cost optimization of several benchmark.

To improve the efficacy of the computational programming of gradient based techniques, Tolson et al. (2009)⁸⁶ develop a hybrid discrete dynamically algorithm, baptised as Form, which simplifies Monte Carlo simulation, and that can be used for determining the critical node location. This algorithm means a reduction in computation time by half, in comparison with the variant of the "Particle Swarm Optimization (PSO) algorithm" developed by I. Montalvo et al (2008)⁵⁴; or the Ant Colony Optimization (ACO) Technique developed by Zecchin et al. (2007)¹⁰⁰. This Ant Colony Optimization Technique has a single objective, which is minimising design costs, and for that Zecchin et al. test out five different (ACO) algorithms, proving them to be very successful for the design of water supply systems, although time consuming too. Nevertheless, none of the previous authors tried varying loads in their algorithms. In order to assess the varying loads issue, R. Dziedzic et al. (2016)¹³ utilise a gradient based procedure to minimise costs as well as expected annual damage of a long-term network with varying loads using shorter time cycles and to adjust pipe diameters. Gradient based techniques are less common, but have convincingly demonstrated that they can provide near-optimal solutions for water networks, as R. Dziedzic¹³ demonstrates. They are also more intuitive than other design and optimization procedures.

According to I. Goulter (1992)²⁴, most of the optimization models that researchers have developed in the last years remain within the academia and are far from being used in a practical world, not because the models do not work, as demonstrated T.M Walski et al. (1987)⁹³, but mainly due to the difficulties for understanding and using them. Designing water supply system is a very complex duty, and it is dependent on an abundant number of variables. Engineers face a difficult job, and the first thing to do for them from a research point of view, is to simplify the task, by facilitating the understanding and comprehension of the new proposals, whilst still be ambitious in the advancement. Many of the nowadays methods focus on complex programming techniques, but they miss an intuitive perspective that is so needed for its practical use. H. Maier et al. (2015)⁴¹ have discussed this issue in their work: They state that the choice of the algorithm itself is most frequently based on the level of familiarity of the author, which is reasonable, but without much comparison between methodologies. Which hinders the process of selecting the most valuable result that can serve as actual guidelines for the future. So far, there is no universal method to optimise the design of water supply systems.

1.3 Key variables

A key variable for operating costs is the efficiency of the pump. Results can vary significantly depending on pump efficiency μ_B ; In fact, μ_B has a wide range of possible values that typically ranges from 70% to 90%. This means a variability of almost 20% in the cost estimate, which is a substantial difference.

Granados System

One of the most used methods to design water supply systems in Spain is the Granados' System (A. Granados (1995)²⁸; F. J. Martín Carrasco and L. Garrote (2013)⁴⁶), which is a gradient based procedure. The logic behind Granados' System is to find which of the following options is cheaper: It could be selecting a small but cheap pipe that will mean high head losses and therefore big energy costs; or on the contrary, it could be selecting a big and expensive pipe that will minimise head losses and reduce energy expenses during the operation life. This intuitive reasoning is what has made Granados' System so successful and used in practice. For this reason, Granados' System is the stone on which this thesis is built. I. Granados (2015)²⁶ analyses in depth influence that changes in the design variables mean over the dynamic programming algorithm of Granados' System.

Among all the variables that affect the Granados System, the efficiency of the pump is the key factor in calculating the diameter of the hydraulic line. One of the objectives of this document is to define the relationship between the efficiency of the pump and the other parameters involved in the procedure, such as the flow rate, the pumping head, the required power, etc. This analysis of pump efficiency not only serves the purposes of the Granados' System, but it can also help in many other methods that require the value of the pump efficiency. This is the case of A. Ostfeld (2005)⁵⁵, where they use an estimated value of 75% in their calculations; for 15, whose values vary from 81% to 84%; J. Gessler and T. Walski (1985)²⁰

Figure 1.1. Alfredo Granados and his daughter Isabel Granados, both Polytechnic University of Madrid professors in Water Resources Management, in 2017 in Murcia. (Photo taken by the author)

estimates the efficiency at 75% and so does E. Alperovits and U. Shamir (1977)¹ and R. E. Featherstone (1983)¹⁴.

On the other hand, because operating costs are spread over the life of the facility, they must be calculated at the design stage. The conversion is calculated using a cumulative factor that depends on the duration of the operation of the water facility, the duration of the construction period and the discount rate used. Regarding the useful life of the installation, some methodologies consider an operating life of 20 years R. Dziedzic (2015)¹³ or J. H Kim (1994)³⁶, but other publications such as W. Wu (2009)⁹⁵, C. S. Stokes (2015)⁸⁴ or W. Wu et al. (2009)⁹⁶ prefer to carry out the analysis for 100 years. In order to have a more accurate evaluation of the life span of a water supply system, R. Ramirez et al. (2020)⁶⁴ provide a complete guide on how to determine the service lifespan of network pipes, comparing their procedure to the Weibull proportional hazard model. This is key when assessing not only energy costs derived from the operation, but also, maintenance and repairing costs, for a more complete assessment (specially in multi objectives methodologies). R. Cobacho et al. (2019)⁸ also focuses on providing an accurate estimation of the expected lifespan of water supply systems, through a simple complexity model that does not require extensive historical database.

Other studies have decided to optimize the design using as a key variable various and diverse factors, such as the location of the pump, like G. Dandy (2000)¹² or G. Tsakiris (2004)⁸⁷, the capacity of the pump as D. Kang (2012)³³, the pump type, like C. S. Stokes et al. (2015)⁸³, the power as J. H. Kim (1994)³⁵, the pumping head like G. M. Gonçalves (2014)²³; J. Marques (2014)⁴³ and A. Ostfeld (2008)⁵⁶, the pumping program like A. Ostfeld (2005)⁵⁵ or A. Ostfeld (2008)⁵⁶; or pressure like D. Vicente et al. (2015)⁹¹ or B. Capelo (2017)⁵. The design of water supply systems is a complex task involving many variables, as pointed by G. Tsakiris (2017)⁸⁸, interconnected with each other, and decision making needs a complete understanding of how each factor affects the installation. Some of the relationships between variables are still unknown, and are only intuited by experience. For this purpose, sensitivity analysis is necessary. G. Fu et al. (2011)¹⁶ performs an exhaustive sensitivity analysis using the Sobol method (an approach based on variance) to determine which variables affect the installation the most. These key variables can go from pipe diameters up to tank sizes, and the one that has the greatest influence depends on each case study. In G. Fu et al. (2011)¹⁶ work they demonstrate how much computational savings can be obtained and, therefore, how beneficial the analysis is. This document also intends to serve as a sensitivity analysis of how much pump efficiency is influenced by other factors,

and how pump efficiency can affect the ultimate cost of a water supply installation.

1.4 Water demand variability

On the other hand, as L. Romero et. al (2017)⁶⁸; A. Sordo-Ward et al. (2016)⁷⁸; A. Sordo-Ward et al. (2019)⁷⁷; A. Iglesias et al. (2018)²⁹; D. Santillán et al. (2019)⁷²; D. Santillán et al. (2019)⁷³; D. Santillán et al. (2019)⁷⁴ and L. Garrote et al. (2014)¹⁸ state, due to climate change, population migrations, etc. variations in demand of water are becoming more extreme, increasing the variability throughout the year, as shown in M. Spiliotis (2019)⁷⁹; B. de Lama (2011)³⁸; L. Garrote (2008)¹⁹; A. Kuriqi et al. (2017)³⁶; A. Kuriqi et al. (2019)³⁷; Mediero. L. et al. (2014)⁵³; F. J. Martín Carrasco (2006)⁴⁵ and M. Spiliotis (2018)⁷⁸. These seasonal variations make it even more complex for designers to properly define efficient water supply systems. For these reasons, design methodologies that can handle changes in demand are desirable, as stated by G. Tsakiris (2017)⁸⁸ and M. Spiliotis (2017)⁸¹. A. Babayan et al. (2005)⁶⁸ present this problem in their optimization using the standard deviation in the demand calculation. In studies such as A. Granados et al. (2015)²⁸ or A. Granados (2013)²⁵ et al. they use safety coefficients that are determined after a sensitivity analysis of the variables that influence the variation in demand, especially for agricultural purposes. A different approach is used in Z. Kapelan (2005)³⁴, where robustness is maximized by generating random flow situations and calculating the probability of meeting the required conditions at all nodes. A. Babayan (2007)² compares the previous approach to the perspective of the safety coefficient making use of a random demand that adjusts to a certain probability, and concludes that although both points of view have similar results, the second one is computationally more expensive.

Regarding the uncertainty in the demand, Walski (2001)⁹⁴ explains an interesting phenomenon that can be described as a “self-prophecy effect from the construction of pipes”: He states that “If you build it, they will come (within reason). What this means in terms of pipe network design is that, as long as the pipe is not undersized the day it is installed, the distribution system will “grow into” the pipe. If water supply capacity is not available where an industry or developer wants to locate, then they will either pay for capacity or find a location where distribution capacity is available”. From this perspective, future demands are actually not unknown, since they become what the pipe allows. This turn around the entire paradigm because, from this point of view, design comes first, and then demand follows afterwards. To minimise the consequences of the “self-

prophecy" effect that Walski warns about, Creaco et al. (2015)¹⁰ advocates to plan the construction of the water supply system in different phases, so that the demand has time to accommodate to the installation and in this way, the design gets adjusted to the uncertainties. This means, they propose "optimizing phasing of construction while accounting for uncertainty in demand growth". Their procedure aims to provide construction planning the maximum flexibility. A similar approach is devolved by Giacomoni (2013)²¹. To confront population growth, land-use change, the hydrologic cycle, residential water use, and inter basin transfers, (which can generate variations in the demand patterns), they propose an adaptive system to confront the dynamic iterations these variations imply. This is done in order to take into account the feedback operation gives back, and to provide adaptations among these systems. In this same line Marques et al. (2015)⁴⁴ propose a methodology to progressively adapt designs that have already been built, to changes in demand, and, in general, to meet the new circumstances that may be required. Their procedure is called Real Options, and they focus on the fact some decisions can be delayed pending future conditions, so that the design gains flexibility and takes less risks when it comes to funds expenditure.

Overall, when it comes to facing water demand variations or infrequent events, in general, the characteristic that better defines a resistant design is resilience. Resilience ensures that the water supply system has the capacity to resist disruptive events. F. Cubillo and A. Martinez Codina (2017)¹¹ test out a resilience model in the case study of Canal Isabel II, to analyse the effectiveness of protocols. R. Martinez (2003)⁴⁹ reviews the different existing procedures to obtain the design flow rate, focussing on Clements' formulation, and analyses how the economic cost of the project is affected by design flow rate variations (caused either by changes in the crops or changes in the service guarantee).

With all this in mind, the objective of this research study is to develop a methodology that can overcome the limitations that a variable demand for water brings to the design of a water supply system and that can be easily understood by facilitating a practical approach. In this document, we propose two new design procedures that take into account variable demand, Equivalent Flow Rate, and Equivalent Volume (A. Martin-Candilejo et al. (2020))⁴⁷. On the other hand, since the pump efficiency can substantially affect the design of water supply systems due to the wide range of values they can adopt, this research carries out an extensive study of the pump performance in order to characterize its values. For this to be made, the features of commercial pumps available in the market are analysed (A. Martin-Candilejo et al. (2019))⁴⁸. The conclusions are integrated in a

gradient based procedure, using as a base the Granados System, giving rise to an optimized design method that accounts for the construction and operational costs. This procedure is also compatible with adding other variables, as multi objective methods do. Gradient based procedures are very intuitive and not very expensive from a computational point of view. This facilitates its use in the practical world, which is, in the end, the main objective of this research. To illustrate the application of this new methodology and its computationally straight forward conception, a case study is presented.

II. Methodology

2.1. Approach

Loosely speaking, the total cost of a hydraulic impulsion is the sum of the cost of construction of the pipe, the cost of construction of the pumping station and the cost of the energy required for the pumps to work during the entire lifespan of the system. Other costs, such as maintenance and operation, may be neglected since they do not depend directly on pump or pipe sizing.

$$\text{Total Cost} = \text{Pipe construction} + \text{Pumping Station Construction} + \text{Energy}$$

Pipe Construction Cost increases as the pipe diameter widens, since the price of the tubes increases, and so too does the excavation and installation cost due to the difficulties that arise through transportation and assembly.

Construction costs of the pumping station can be considered relatively independent of that of the chosen pipe, since the different pump models usually have a similar cost and the group and stage configurations depend on the variability of the demand and not on the pipe features.

The cost of energy depends, basically, on the volume of water to be elevated, but also, on the head and the pump efficiency. The head depends on the diameter of the pipe, since the larger the pipe diameter, the less head loss. In this way, using a larger diameter reduces the pumping cost. The efficiency of the pump is a variable whose relationship with the other factors had not been studied in depth until recently. As a general rule, it was assumed that the larger the pump is, the better performance it had. In this document this issue is studied in depth in future chapters.

The following **figure 2.1.** schematically shows how the cost distribution would vary depending on the pipe diameter. Since the pipes diameters do not form a continuous series, but instead, are discrete values in the market, the traditional way of solving the problem has consisted of proposing various alternatives of diameters and pumps that meet the technical requirements of the problem, then each alternative is evaluated, and the one with the lowest cost is selected.

Figure 2.1. Simplified scheme of the costs of the water drive depending on the diameter $\varnothing$ of the pipeline: As the diameter increases the construction costs rise but the head losses decrease, implying a reduction in the energy cost.

The procedure proposed in this work is based on the following principle: The head loss can be reduced by increasing the pipe diameter. This means savings in the energy cost but also an increase in the cost of construction. This can be studied from a unitary perspective: Comparing the cost of reducing one meter of head loss by increasing the pipe diameter with the savings in energy expenditure by not having to elevate water that one extra meter. This procedure uses the **Change Gradient** concept that was developed by Professor Alfredo Granados as part of his pipe network optimization method, the **Granados' System**. But the Change Gradient only makes sense when the flow rate is constant. Therefore, for more realistic situations, this paper has developed the new concepts of the Equivalent Flow Rate and the Equivalent Volume for two different procedures for the design of a hydraulic drive.

2.2. Change Gradient Concept

The Change Gradient is defined as the cost of reducing one meter of head loss by increasing the pipe diameter from ϕ_i to the next bigger one $\phi_j = \phi_{i+1}$:

$$GC_{\phi_i \rightarrow \phi_j}^q = \frac{p_j - p_i}{\Delta h_i^q - \Delta h_j^q}$$

Being:

- ▶ p_i, p_j are prices of pipes of length L and diameters ϕ_i and ϕ_j , respectively.
- ▶ $\Delta h_i^q, \Delta h_j^q$ are the head losses of a pipe of length L and diameters ϕ_i and ϕ_j for a given q flow rate.

This is represented in **figure 2.2**.

Figure 2.2. Change Gradient concept. Increasing the pipe's diameter means a reduction in the head loss along the pipeline, but it has the extra cost of the wider and more expensive tube. The Change Gradient is the cost of reducing the head loss by 1 meter.

If, by instance, the head loss is calculated using Manning's formulation, the length of the pipe is cancelled out from the Change Gradient's expression, and it stands as:

$$GC_{\phi_i \rightarrow \phi_j}^q = \frac{p_j L - p_i L}{\frac{L n_i^2 2^{20/3} q^2}{\pi^2 \phi_i^{16/3}} - \frac{L n_j^2 2^{20/3} q^2}{\pi^2 \phi_j^{16/3}}} = \frac{\pi^2}{n^2 2^{20/3}} \frac{(p_j - p_i)}{\left(\frac{n_i}{\phi_i^{16/3}} - \frac{n_j}{\phi_j^{16/3}} \right)} \frac{1}{q^2} = K_{GC} \frac{1}{q^2}$$

being K_{GC} :

$$K_{GC} = \frac{\pi^2}{n^2 2^{20/3}} \frac{(p_j - p_i)}{\left(\frac{1}{\phi_i^{16/3}} - \frac{1}{\phi_j^{16/3}} \right)},$$

Where:

- ▶ L is the pipe length, p_i and p_j are the prices of the pipes of a diameter ϕ_i and ϕ_j , respectively.
- ▶ n_i and n_j are the Manning coefficients for the roughness of pipes i and j , respectively.
- ▶ q is the flow rate through the pipe.

Regarding the available diameters, each pipe manufacturer only offers a finite number of commercial diameters for each type of pipe. This means that the designer must adjust to the series of diameters offered for that pipe type.

Parallel drive

Figure 2.3. Parallel pipe configuration of the hydraulic drive.

Sometimes the pipes are conformed by two or more twin pipes, usually connected in parallel, that start from the same point and reach the same destination, as figure 2.3. represents. In these cases, the flow rate through the drive q is distributed between these pipes, passing through each of them q/n_t , where n_t is the number of equal pipes that make up the drive. The price of these pipes is n_t times the price of an independent pipe. The Change Gradient corresponding this type of situations follows the following expression:

$$GC_{\phi_i \rightarrow \phi_j}^{q/nt} = \frac{\pi^2}{n^2 2^{20/3}} \frac{nt (p_j - p_i)}{\left(\frac{1}{\phi_i^{16/3}} - \frac{1}{\phi_j^{16/3}} \right)} \frac{1}{(q/nt)^2} = nt^3 GC_{\phi_i \rightarrow \phi_j}^{q/1t}$$

2.2.1. Energy Cost

During the operation of a pumping station it is common that different discharges are pumped to meet the needs of the demand. The energy consumed in each of these situations is different, depending not only on the flow pumped, but also on the height pumped, the electromechanical efficiency, the duration of this pumping situation and the unit price of the energy.

If a simple case is analyzed, it can be assumed that there are n pumping situations that differ by the flow rate pumped ($q_1, q_2, q_i \dots q_n$). In this case, the annual cost for the energy consumed c_E can be obtained from the following expression (the first and second equality are in International System units, whilst the third one depends on the following chosen units):

$$c_E = \sum_1^n E_i p_i = \sum_1^n \gamma q_i h_i \frac{1}{\mu_{B_i}} \frac{1}{\mu_{M_i}} t_i p_i = \sum_1^n 9,81 \frac{V_i}{3600} h_i \frac{1}{\mu_{B_i}} \frac{1}{\mu_{M_i}} p_i ,$$

where

- ▶ c_E is the annual cost of energy consumed by pumping (€).
- ▶ E is the annual energy consumed in the situation i (kWh).
- ▶ p_i is the unit price of energy in the situation i (€/kWh).
- ▶ γ is the specific weight of the pumped liquid (for water 9.810 N/m^3).
- ▶ q_i is the discharge pumped in the situation i (m^3/s).
- ▶ h_i is the height pumped in the situation i (m).
- ▶ μ_{B_i} and μ_{M_i} are the pump and engine efficiency in the situation i respectively.
- ▶ t_i is the annual duration of situation i (hours).
- ▶ V_i is the annual volume (m^3) pumped during situation i .

This c_E cost is repeated every year during the lifespan of the pipe. To obtain the capitalized cost, the annual costs must be integrated at present value into a single global energy cost. The total cost of the energy spent during the useful life of the pipes capitalized at the beginning would therefore be C_E :

$$C_E = f_A \cdot c_{E1}$$

f_A is called discount factor and it depends mainly on the discount rate i , the lifespan of the pipe n_u and the construction period duration n_c . A generalist definition of f_A would be:

$$f_A = \frac{(1+i)^{n_u} - 1}{(1+i)^{n_u} i} \cdot \frac{1}{(1+i)^{n_c}}$$

2.2.2. Optimization for a constant flow rate.

Granados System

This situation is given when the pumping station works with a constant flow rate. This means the operating point of the pump does not change. In this way, all the other variables that affect the energy cost (q_i , h_i , μ_B , μ_M y p_i) are constant. In this case, the flow rate q , can be calculated, as well as the Change Gradients $GC_{\emptyset_i \rightarrow \emptyset_{i+1}}^q$ associated to the increase of diameter $\emptyset_i$ to the next bigger one $\emptyset_j = \emptyset_{(i+1)}$. Likewise, as much as the cost of pumping energy is concerned, since the pump always works at the same operating point, the pump efficiency μ_{Bi} , the engine efficiency μ_{Mi} and the pumping height h_i remain constant (they can be simplified to just h , μ_B , μ_M) and the total cost of energy C_E can be obtained with the following expression:

$$C_E = f_A C_E = f_A \sum_1^n 9,81 \frac{V_i}{3600} h_i \frac{1}{\mu_{Bi}} \frac{1}{\mu_{Mi}} p_i = f_A 9,81 \frac{\sum_1^n V_i}{3600} h \frac{1}{\mu_B} \frac{1}{\mu_M} p.$$

From the previous expression, making $h=1m$ and $\sum_1^n V_i = V$ (which is the total annual volume pumped), the cost of the energy required for each meter of elevation C_{E1} can be calculated as:

$$C_{E1} = \frac{C_E}{h} = f_A 9,81 \frac{V}{3600} \frac{1}{\mu_B} \frac{1}{\mu_M} p$$

With the exception of the pump performance μ_B , the other variables in the previous equation are actually data: V is the total volume to be pumped, p is the unit price of the energy that has been hired, and f_A is calculated from the discount rate i , the service life n_u of the water pipeline and the construction period n_c . Regarding the engine efficiency μ_M , although it varies theoretically depending on the engine model chosen and the operating point of the pump, the variations in engine performance are so small that it can be considered constant across different models and manufacturers (A. Martín Candilejo et al. (2019)⁴⁸).

Therefore, the above equation can be simplified by grouping all the data and parameters that have fixed values in the coefficient KC_{E1} , which can be considered constant for each case analyzed:

$$C_{E1} = KC_{E1} \frac{1}{\mu_B}$$

$$KC_{E1} = f_A 9,81 \frac{V}{3600} \frac{1}{\mu_M} p$$

In the previous expression it is shown that the only variable that affects is the pump efficiency, μ_B , which will depend on the pump model chosen and the operating point (and it is yet unknown).

► The reasoning

For this simple case of constant pumping flow, a design procedure for selecting the pipe diameter $\emptyset$ is established through the following reasoning:

- ◀ If $GC_{\emptyset_i \rightarrow \emptyset_{i+1}}^q < C_{E1}$ → Diameter $\emptyset_{i+1}$ is preferable to $\emptyset_i$ since the cost of reducing 1m the head loss by passing from $\emptyset_{i+1}$ to $\emptyset_i$ is cheaper than the cost of pumping that additional meter.
- ◀ If $GC_{\emptyset_i \rightarrow \emptyset_{i+1}}^q > C_{E1}$ → Diameter $\emptyset_i$ is preferable to $\emptyset_{i+1}$ since the cost of reducing 1m the head loss by passing from $\emptyset_{i+1}$ to $\emptyset_i$ is more expensive than the cost of pumping that additional meter.

If this comparison is started on the diameter $\emptyset_1$, which is the smallest in the series of diameters available for that pipe model and manufacturer (that meets the maximum velocity condition along the network), the process leads to the optimum diameter for the water drive.

► Breaking the loop

But it has already been indicated that the value of C_{E1} depends on the pump model chosen, and more specifically, on its efficiency μ_B at the point of operation. Therefore, theoretically, the pump should first be chosen so that the pipe diameter can be selected. But the μ_B efficiency at the operating point can only be obtained when the operating point is known, which depends on the diameter of the pipe. Therefore, the pipe diameter should first be known to obtain the operating point of the pump and thus its efficiency.

Therefore, starting by either selecting the pump, or the pipe diameter, the process involves iterating to find the optimal pipe diameter for each specific pump model. If the pump model changes, even slightly, the operating point would change and so would the pipe diameter.

In order to break this vicious circle, here we propose a more direct calculation process. It consists on solving the pump efficiency that each diameter requires to be competitive. As it is shown in the following equation, diameter ϕ_i will be the optimum (this means, it should not be substituted by to the next diameter ϕ_{i+1} of the series) when:

$$GC_{\phi_i \rightarrow \phi_{i+1}}^q > C_{E1} \rightarrow GC_{\phi_i \rightarrow \phi_{i+1}}^q > KC_{E1} \frac{1}{\mu_B} \rightarrow \mu_B > \frac{KC_{E1}}{GC_{\phi_i \rightarrow \phi_{i+1}}^q} \rightarrow \text{Keep } \phi_i,$$

Otherwise:

$$GC_{\phi_i \rightarrow \phi_{i+1}}^q < C_{E1} \rightarrow GC_{\phi_i \rightarrow \phi_{i+1}}^q < KC_{E1} \frac{1}{\mu_B} \rightarrow \mu_B < \frac{KC_{E1}}{GC_{\phi_i \rightarrow \phi_{i+1}}^q} \rightarrow \text{Move to } \phi_{i+1}$$

This means that, whenever there is a pump on the market whose efficiency can be greater than the calculated μ_B , the optimum diameter will be ϕ_i . In the event that no commercial pump can reach that performance because it is very high, it will be necessary to move to the next diameter ϕ_{i+1} . Therefore, to apply this method it is necessary to know the maximum performance that pumps can reach. This is hence a major aspect that has been studied in depth in this thesis. It will be covered in the Pump efficiency Analysis section.

Of course, only diameters that meet the minimum limitations required will be selected. These limitations might be maximum velocity, pressure ((D. Vicente (2015))⁷³, (M. Perez-Sanchez (2018))⁷⁴), etc.

2.2.3. Optimization for variable flow rate

During the operation of a pumping station, it is very common to work at different flow rates to adjust to the demand needs. The energy consumed in each situation is different to one another, depending, not only on the flow rate, but also on the height at which water is pumped, the electromagnetic efficiency, the duration of the specific situation, and the energy price during the period of the day.

To illustrate this, the following **figure 2.4.**, two different situations arise for the same pump, where it can be seen that the change in the pumped flow (caused, for example, by starting one of the pumps of the station, or by varying the geometric difference) causes changes in the other indicated parameters.

Figure 2.4. Situations in which the flow rate changes: On the right, activation of additional pumps; on the left, the geometrical difference between the reservoir and the destination deposit increases.

Pumping stations that are designed to work with variable flow rates are more frequent in civil engineering applications, since they allow a better adjustment of the flow rates to those demanded at any time, avoiding in this way the situation of pumping a flow greater than the one required, and therefore, reducing the height losses with the consequent saving of energy. For this more realistic situation, the previously proposed method raises two problems:

- ▶ The Change Gradient can only be calculated at a constant flow rate.
- ▶ The energy cost C_{E1} is calculated for a constant pump performance and head.

To solve these two issues, this study proposes two new methods called the "**Equivalent Flow Rate**" and the "**Equivalent Volume**" methods explained below.

2.2.3.1. Concept and calculation of the Equivalent Flow Rate

Definition

The **Equivalent Flow Rate** q_{Eq} can be defined as a theoretical flow rate for which, if all the volume V required in a year was pumped at this discharge, the cost of the pumping energy would be the same as the cost of pumping at a variable flow rate regime.

Calculation

For the following reasoning it is convenient to use a theoretical example. Assuming that the required annual volume V is pumped through a specific pipe and following a variable flow distribution, the annual energy cost c_E , applying the Equivalent Flow Rate definition, would result in:

$$c_E = \sum_{k=1}^n 9,81 \frac{V_k}{3600} h_k \frac{1}{\mu_{B_k}} \frac{1}{\mu_{M_k}} p_k = 9,81 \frac{V}{3600} h_{Eq} \frac{1}{\mu_{B_{Eq}}} \frac{1}{\mu_{M_{Eq}}} p_{Eq}$$

Being:

- ▶ V the annual volume of water pumped.
- ▶ h_{Eq} the pumping head for the theoretical operating point corresponding to q_{Eq} .
- ▶ $\mu_{B_{Eq}}$ and $\mu_{M_{Eq}}$ the pump and engine efficiency at the theoretical operating point corresponding to q_{Eq} .
- ▶ n the number of periods of different flow rate.
- ▶ p_{Eq} the theoretical unit price of the energy with which q_{Eq} would be pumped.

Figure 2.5. Equivalent Flow Rate definition.

For this analysis, the cost of pumping energy should be separated into two parts. On the one hand, the cost corresponding strictly to the geometric height, and on the other hand, the cost corresponding to the head losses. In this way, knowing that $h_k = h_G + \Delta h_k$, the previous equation would be:

$$\begin{aligned} c_E &= \sum_1^n 9,81 \frac{V_k}{3600} (h_G + \Delta h_k) \frac{1}{\mu_{B_k}} \frac{1}{\mu_{M_k}} p_k \\ &= 9,81 \frac{V}{3600} (h_G + \Delta h_{Eq}) \frac{1}{\mu_{B_{Eq}}} \frac{1}{\mu_{M_{Eq}}} p_{Eq} \end{aligned}$$

where h_G is the geometric height and Δh_{Eq} is the head loss when q_{Eq} is circulating.

Some simplifications can be made in the previous equation. To begin with, the motor efficiency, and the unit price of energy can be assumed as constant. These assumptions are quite close to reality, since, as it has been already indicated, motor efficiencies vary very little in different operating regimes, and it is common to hire a flat rate of energy. With the above simplifications, the expression would be:

$$V h_G + V \Delta h_{Eq} = \sum_1^n \frac{\mu_{B_{Eq}}}{\mu_{B_k}} V_k h_G + \sum_1^n \frac{\mu_{B_{Eq}}}{\mu_{B_k}} V_k \Delta h_k.$$

Now, a new simplification must be made, and this one is more debatable. It consists on assuming a constant pump efficiency. This implies that all μ_{B_k} are equal, including $\mu_{B_{Eq}}$. This is a purely operational assumption and does not correspond to reality, since the pump's performance varies significantly for the different operating points. This simplification is made in order to prevent from future iterations to obtain the Equivalent Flow Rate, and it is later shown to be unnecessary.

The general expression would be:

$$V h_G + V \Delta h_{Eq} = \sum_1^n V_k h_G + \sum_1^n V_k \Delta h_k \quad \rightarrow \quad V \Delta h_{Eq} = \sum_1^n V_k \Delta h_k.$$

On the other hand, the various head losses associated to different flow rates can be expressed as below using Manning's expression:

$$\Delta h_k = \frac{L n^2 2^{20/3} q_k^2}{\pi^2 \phi^{16/3}} = \alpha q_k^2$$

Applying this to the previous equation, the new general expression is:

$$V \propto q_{Eq}^2 = \sum_{k=1}^n V_k \propto q_k^2,$$

and it can finally be simplified to obtain the **Equivalent Flow Rate**:

$$q_{Eq} = \sqrt{\frac{\sum_{k=1}^n V_k q_k^2}{V}}$$

Where:

- ▶ q_{Eq} is the Equivalent Flow Rate. It is a constant flow that implies the same energy cost as that of a variable flow regime.
- ▶ q_k is the flow pumped in the operating situation k .
- ▶ $V_k = q_k \cdot t_k$, is the annual volume in operating situation k .
- ▶ t_k is the annual time spent in pumping q_k .
- ▶ V is the annual volume pumped, calculated as $V = \sum_{k=1}^n V_k$.

The previous expression shows that the Equivalent Flow Rate is an average flow rate of the variable regime. From this point of view, it is an intermediate discharge that should correspond to an operating point close to the optimum of the pump, in a way that the real variable flow values are located around the sides of this optimum. (A. Martín-Candilejo et al. (2020))⁴⁸

It should be remembered that the previous equivalent flow rate has been obtained by making three assumptions: that the engine efficiency, that the unit price of energy, and that the pump efficiency are the same for any operating point. The first two are logical and respond to the reality of most cases. The third one is, however, conceptually incorrect and it would theoretically require a second approximation of the Equivalent Flow Rate once the pipe diameter has been obtained and the pump selected. However, our simulations let us affirm that this first approximation of the equivalent flow rate is already sufficient, since the pump performances, although different, compensate between the different operating points if the pump is properly selected.

2.2.3.2. Concept and calculation of the Equivalent Volume

Concept

If a constant flow rate q_m is chosen to design the pipe line (preferable $q_m = 1 \text{ m}^3/\text{s}$ to make calculations easier), $V_{Eq}^{q_m}$ is the total Equivalent Volume of water that

needs to be pumped at that flow rate q_m to make the final cost of pumping equal to what it would be pumping at a variable regime of flow rates.

Calculation

For the calculation of the Equivalent Volume $V_{Eq}^{q_m}$, a similar reasoning to the one followed for the Change Gradient will be applied: The final cost will always be referred to the cost of elevating the water at 1m height C_{E_1} . Applying this approach in the previous equation:

$$C_{E_1} = \frac{C_E}{h} = f_A 9,81 \frac{V}{3600} \frac{1}{\mu_B} \frac{1}{\mu_M} p$$

C_{E_1} can be expressed in terms of $V_{Eq}^{q_m}$ as below:

$$C_{E_1} = f_A 9,81 \frac{V_{Eq}^{q_m}}{3600} \frac{1}{\mu_B} \frac{1}{\mu_M} p = K_{C_V} \frac{V_{Eq}^{q_m}}{\mu_B}$$

Being K_{C_V} :

$$K_{C_V} = f_A 9,81 \frac{1}{3600} \frac{1}{\mu_M} p.$$

Therefore, the concept of the Equivalent Volume is expressed as follows:

$$C_{E_1} = \sum C_{E_1}^{q_k} = C_{E_1}^{q_m} = K_{C_V} \frac{V_{Eq}^{q_m}}{\mu_B}$$

Apart from that, in the situation that a flow rate q_i is circulating, an increase of the pipe diameter from ϕ_i to ϕ_{i+1} would mean a reduction of the head loss $\Delta\Delta h_{\phi_i \rightarrow \phi_{i+1}}^{q_k}$ that, using Manning's expression, only depends on the flow discharge q_i . To simplify the terminology $\Delta\Delta h_{\phi_i \rightarrow \phi_{i+1}}^{q_k}$ will be referred as $\Delta\Delta h^{q_k}$.

$$\Delta\Delta h_{\phi_i \rightarrow \phi_{i+1}}^{q_k} = \Delta\Delta h^{q_k} = \Delta h_{\phi_i}^{q_k} - \Delta h_{\phi_{i+1}}^{q_k} = \beta \cdot q_k^2$$

being β a constant deduced from the invariable terms in Manning's formula.

This reduction of the head loss translates in a savings in the energy cost $C_{\Delta E}^{q_k}$. To express $C_{\Delta E}^{q_k}$, the same simplifications that were used for the Equivalent Flow Rate will be made: the engine efficiency, that the unit price of energy, and that the pump efficiency are the same for any operating point. Once again, the first two assumptions are close to reality, but the third one is not; however further simulations have shown that the pump performances, although different, compensate between the different operating points. This being said, the savings in energy costs can be expressed as it follows.

$$C_{\Delta E}^{q_k} = f_A 9,81 \frac{V_k}{3600} \Delta \Delta h^{q_k} \frac{1}{\mu_{B_k}} \frac{1}{\mu_{M_k}} p_k = K_{C_V} V_k \Delta \Delta h^{q_k} \frac{1}{\mu_B}$$

Additionally, any variance of the head loss associated to a flow rate can be expressed in terms of a different discharge as:

$$\Delta \Delta h^{q_k} = \beta \cdot q_k^2 \rightarrow \Delta \Delta h^{q_k} = \beta \frac{q_m^2}{q_m^2} q_k^2 = \Delta \Delta h^{q_m} \frac{q_k^2}{q_m^2}$$

Introducing the previous expression in the equation of $C_{\Delta E}^{q_i}$, it is then:

$$C_{\Delta E}^{q_k} = K_{C_V} V_k \Delta \Delta h^{q_m} \frac{q_k^2}{q_m^2} \frac{1}{\mu_B}.$$

However, as it was expressed in the Equivalent Volume definition, this needs to be analyzed from the perspective of pumping water at 1m height so that, later on, the Change Gradient methodology can be followed. For this unitary point of view, $C_{E_1}^{q_i}$ is obtained from $C_{\Delta E}^{q_i}$ by:

$$C_{E_1} = \frac{\sum C_{\Delta E}^{q_k}}{\Delta \Delta h^{q_m}} = K_{C_V} \frac{\sum V_k q_k^2}{q_m^2} \frac{1}{\mu_B}.$$

Introducing this conclusion in the equation written in the definition, the Equivalent Volume formulation can be deduced:

$$K_{C_V} \frac{V_{Eq}^{q_m}}{\mu_B} = K_{C_V} \frac{\sum V_k q_k^2}{q_m^2} \frac{1}{\mu_B}.$$

Therefore, the Equivalent Volume is:

$$V_{Eq}^{q_m} = \frac{\sum V_k q_k^2}{q_m^2}.$$

This expression is simplified if the virtual constant discharge of design q_m takes the value of $q_m = 1\text{m}^3/\text{s}$. For this value, the **Equivalent Volume** is simplified as:

$$V_{Eq}^{q_m=1\text{m}^3/\text{s}} = \sum V_k q_k^2$$

2.3. Analysis of the pump efficiency

Among all variables affecting Granados System, the pump performance is the key factor for calculating the diameter of the hydraulic conduction. Nevertheless, the results may vary significantly depending on the pump efficiency; and it is that, indeed, μ_B presents a wide range of possible values that typically goes from 70% up to 90%. This means a variability of almost 20% in the estimation of the cost, which is a substantial difference. Therefore, the aim of this work is to define the relationship between the pump's performance and the other parameters involved in the procedure, such as the flow rate, the pumping head or the required power.

For this to be done, we select 400 commercial pumps from the catalogues of several manufactures. We discard custom made pumps since the cost of this pumps is much higher than the ordinary one's listed in manufactures series and the offer in commercial catalogues is already very wide. The selected pumps vary from each other in their type, impeller diameter, number of stages, rotation speed (electrical current frequency, number of poles), brand, etc. For all this models, we study the pump efficiency, in particular, we register the optimum value together with the correspondent flow rate, head, and power consumed for it. Nevertheless, some pumps are ruled out of the sample and presentation of the results because they either were repetitive along the manufactures, or because they were very specific for some industrial or sanitary engineering uses. In the end, the sample consists of 226 hydraulic pumps.

Description of the sample

For the detailed study of the pump performance, this research has focused on the most common type of pump for the applications in civil engineering (supply, irrigation, sanitation, etc.), which is centrifugal pumps. Within centrifugal pumps, both horizontal and vertical axed are selected, mostly with a radial flow configuration, with the exception of submersible pumps, for which the axial arrangement is more common. The sample includes regular horizontal and vertical pumps, split case pumps, multistage and submersible pumps. The manufacturers used for this analysis was IDEAL, WILO, ESPA and HASA. More specifically, the commercial models were:

- ▶ Split case pumps: CP/CPI/ CPR series.
- ▶ Horizontal pumps (normalized in the European Union): RNI/RN series.
- ▶ Multistage horizontal pumps: APM series.
- ▶ Vertical pumps: VS/VG series.

- ▶ Submersible vertical pumps: SVA/SVH series.

Multistage pumps perform with the same efficiency for a specific flow rate and different heights (which is the number of stages multiplied for the unitary head). To avoid this dispersion that could make it difficult to draw conclusions, it was decided to only use the optimum operating point correspondent to a single stage.

Using the data collected for the optimum operating points for all the 226 hydraulic pumps previously mentioned (pump's optimum efficiency and associated flow rate, head, speed, power, frequency, diameter, etc.) we carried out an analysis to establish relationships among the design variables of a water drive.

III. Results and Discussion

3.1. Pump efficiency analysis results

The objective of the pump efficiency analysis was to determine an indicative value of the optimum μ_B to expect. Using the collected data from the pumps sample, different variables are plot against each other, so that it can be observed if there is any relationship between μ_B and any of the other factors involved in the design of a water supply system.

3.1.1. Empiric results and discussion

3.1.1.1. Relationship between pump efficiency and flow rate

Figure 3.1. shows the values of the pump's optimum efficiency and the flow rate correspondent to that point. From this representation, it can be seen that there is a relationship between both variables: the performance of the pump improves as the discharge increases, although it seems to find a horizontal asymptote for μ_B around 90%. In this same figure, it can be spotted that the relationship between the two variables is more precise for flow rate values that are greater than 500 l/s. Below this value, as it can be seen in **figure 3.1. (b)**, there is more dispersion. Also, when the discharge is at least 100 l/s, the pump efficiency can be expected to be better than 80%, but under it, the dispersion is stronger. **Figure 3.1. (c)** shows the same results in logarithmic scale, for a better visualisation.

(a)

(b)

(c)

Figure 3.1. Relationship between the flow rate and the pump efficiency. (a) Distribution up to 300 l/s, and (b) amplified figure for up to 500 l/s. (c) Logarithmic scale for a better visualization.

3.1.1.2. Relationship between pump efficiency and the pumping head

When the pumping height and the efficiency are plot together, as show in **figure 3.2.** there is no clear sign of a relationship between the two variables since the dispersion is too strong for any head value. There is a very light tendency of high pump performances (around 80%) for under 40 m head, but over 60m the distribution of μ_B is too scattered.

Figure 3.2. Relationship between the pumping head and the pump efficiency.

3.1.1.3. Relationship between pump efficiency and the pumping power.

To test out what the relationship between the optimum μ_B and the required power for that operating point is, **figure 3.3.** was elaborated.

(a)

(b)

Figure 3.3. (a) Relationship between the pumping power and the pump efficiency. (b) Same figure, in logarithmic scale.

The main conclusions drawn from this figure are:

- ▶ There is a relationship between the pump efficiency and the required power for the optimum operating point.
- ▶ The relation seems to be clearer from 400 kW and on, and values greater than 80% efficiency can be expected.
- ▶ This relationship seems to be weaker than the existing one between the flow rate and the pump efficiency. The interpreted reason for this fact is that, since the power is calculated from the flow rate and the pumping height, the good relationship that the flow rate transfers to the power is weakened by the poor bond that the pumping head and efficiency have (J.H. Mays (1994))⁴. had already thrown out similar conclusions. In their work, they empirically proved that the cost of pumping depended on power; which indeed agrees with this results.

3.1.1.4. Effect of the pump type on the pump efficiency

This first assessment concludes that the strongest of the relationships with the pump efficiency is that of the flow rate, especially when the flow is greater than 100 l/s and it is almost lineal. Therefore, effect of the pump type will be shown with respect to the flow rate-pump efficiency relationship.

The following **figure 3.4. (a)** shows the distribution of the operating points for the split case pumps, horizontal and multistage (both horizontal and vertical). To avoid agglomeration and facilitate the interpretation of **figure 3.4. (b)** shows again the relationship between the different pump types and the pump efficiency for discharge values under 200 l/s.

(a)

(b)

(c)

Figure 3.4. Optimum pump efficiency and the correspondent flow rate at that operating point, classified by the pump type. Detail of the previous figure for smaller discharges. (c) Results in logarithmic scale for a better visualization.

From this graph we conclude that:

- ▶ Split case pumps are, in general, the ones that provide the best performance for flow rates over 500 l/s.
- ▶ Between 100 l/s and 500 l/s, both split case and horizontal offer the best results.
- ▶ Under 100 l/s the distribution is very heterogeneous and disperse, but as a general rule, there is always a horizontal pump that can provide the best efficiency (but also the lowest, due to the dispersion). Vertical pumps also show great performances, but it can generally be exceeded by a horizontal model.
- ▶ Vertical multistage pumps always give a better efficiency than horizontal multistage pumps.
- ▶ For small flow rates, the differences between the pump efficiency among models is very high. Different models can almost double other pump's efficiency, meaning that the energy cost can be almost twice as much if the pump is not well selected. This is why the pump analysis is more complicated for small discharge values.

3.1.1.5. Effect of the pump speed on the pump efficiency

Rotation speed

On the other hand, the relationship between the rotation speed of the pump and the efficiency is shown in **figure 3.5**. As it was expected, the results are well separated depending on the number of poles of the engine. The most common values have been: For one pole, the asynchronous speed is 2900 rpm, and for two poles, 1450 rpm (European frequency of 50 Hz). Most pumps belong to these two categories, showing no apparent relationship with the pump performance, but instead, a wide interval of possible values.

Figure 3.5. Optimum pump efficiency and the correspondent rotation speed of the pump.

Specific speed

Different conclusions can be obtained from the study of the specific speed of the pump, defined as the following expression:

$$n_s = \frac{n_r \cdot q^{1/2}}{H^{3/4}}$$

Being:

- ▶ n_s the specific speed.
- ▶ q the flow rate at the optimum operating point.
- ▶ H the pumping head at the optimum operating point.
- ▶ n_r the rotation speed at the optimum operating point [39].

The specific speed is interpreted as the rotation speed that a homologous pump should have in order to elevate a discharge of 1 m³/s at 1m height, according to M. Pérez-Sánchez et al. (2018)⁷⁶.

Low specific speed indicate that the pump is suitable for small flow rates and great heights. This means that, since small flow rates have a poorer relationship with the pump efficiency, pumps with a low specific speed will have the worst performance values. This can be confirmed in **figure 3.6**. Per contrary, high specific speed is an indicator of a pump suitable for great flow rates and low heights.

According to the previous analysis made in the previous sections **figure 3.1**., greater discharges correspond to better performances, and therefore, a high specific speed can be associated with a good μ_B , as **figure 3.6**. shows. The scattering of the figure is caused by the poor relationship between the pumping height and the rotation speed, and the pump efficiency. However, there is a diffuse tendency of increasing μ_B as the specific speed grows.

(a)

(b)

Figure 3.6. (a) Optimum pump efficiency and the correspondent specific speed of the pump.
(b) Results in log scale.

As expected, the types of pumps that operate at higher flow rates will be the ones with a greater specific speed and therefore, better performance: These are split case pumps, as it is shown in **figure 3.7**. Vertical multistage pumps also give good results in the smaller flow rate ranges.

Figure 3.7. Optimum pump efficiency and the correspondent specific speed of the pump, classified by the pump type.

3.1.2. Mathematical adjustment

From the previous figures, it seems that there are three flow rate ranges, where the relationship with the pump efficiency is different:

- The first one would be from 0 to 100 l/s.
- The second from 100 to 500 l/s.
- The last one over 500 l/s.

Nevertheless, in order to be more precise, instead of three zones, the curve has been fitted using up to fourteen flow rate subdivisions. **Table 3.1.** shows the average optimum pump efficiency for each interval, as well as the maximum and minimum found among the studied pumps.

Elaboration of the adjusted curves of the pump efficiency versus the flow rate

Pump efficiency for each flow rate interval						
Interval number	Flow rate range			Pump efficiency		
	Minimum q (l/s)	Maximum q (l/s)	Average q (l/s)	Average μ_B (%)	Maximum μ_B (%)	Minimum μ_B (%)
1	0	5	2.5	55.3%	77.0%	36.2%
2	5	10	7.5	63.5%	76.0%	45.0%
3	10	15	12.5	65.7%	77.0%	45.0%
4	15	25	20	74.2%	82.0%	63.0%
5	25	40	32.5	75.0%	85.0%	61.2%
6	40	65	52.5	78.6%	84.0%	70.5%
7	65	100	82.5	81.0%	85.4%	75.4%

8	100	150	125	83.1%	86.0%	74.0%
9	150	200	175	82.8%	87.0%	80.4%
10	200	300	250	83.9%	88.0%	81.0%
11	300	500	400	84.9%	88.8%	79.9%
12	500	1.000	750	85.5%	88.5%	80.4%
13	1.000	2.000	1.500	88.3%	89.0%	86.7%
14	2.000	3.000	2.500	88.2%	89.0%	86.2%

Table 3.1.

Adjusted curves

When these values are adjusted through a doubly logarithmic curve, the relationship for both, the average and maximum values, fits satisfactory ($r^2 > 98\%$ and $r^2 > 90\%$ respectively), as it can be seen in **figure 3.8**. The empirical equations that relate the optimum pump efficiency and the flow rate are:

$$\mu_B^{\text{Average}} = 0.1286 \ln (2.047 \ln q - 1,7951) + 0.5471 \quad r^2 > 98\%,$$

$$\mu_B^{\text{Maximum}} = 0.0576 \ln (2,047 \ln q - 1.7951) + 0.741 \quad r^2 > 90\%,$$

being q the flow rate in litres per second (l/s).

Figure 3.8. Adjusted curves. The curves are elaborated using 14 intervals of the collected data.

Finally, **figure 3.9** shows the adjusted curves along with the collected data: Both curves fit satisfactory for most flow rate ranges, especially for the bigger ones, nevertheless the dispersion is higher for the small discharge values (under 50 l/s), but still, it gives a valid reference. To facilitate the visualization, **figure 3.9** (b) also shows the results in a logarithmic scale.

(a)

(b)

Figure 3.9. (a) Optimum pump efficiency curve: Data base and mathematical adjustment. (b) Same curves in logarithmic scale for better visualization.

3.2. Procedure Assembled

3.2.1. Optimization procedure using the Equivalent Flow Rate Method

The Equivalent Flow Rate has been developed to calculate the Change Gradients when the discharge regime is variable. As previously explained, each Change Gradient is associated with a specific flow rate. Therefore, when the flow regime is variable, it is necessary to use the Equivalent Flow Rate to calculate them. In this way, the expression of the Change Gradient will change to:

$$GC_{\phi_i \rightarrow \phi_j}^{q_{Eq}} = \frac{P_j - P_i}{\Delta h_i^{q_{Eq}} - \Delta h_j^{q_{Eq}}}.$$

Using the concept of the Equivalent Flow Rate in the calculation of the cost of energy per meter lifted, the previous equation turns into:

$$C_{E1} = \frac{C_E}{h_{Eq}} = f_A \cdot 9,81 \cdot \frac{V}{3600} \cdot \frac{1}{\mu_{B_{Eq}}} \cdot \frac{1}{\mu_{M_{Eq}}} \cdot p_{Eq} = KC_{E1}^{q_{Eq}} \cdot \frac{1}{\mu_{B_{Eq}}}.$$

It has already been discussed that the engine efficiency and the unit price of energy can be assumed equal for any operating point. For this reason, coefficient $KC_{E1}^{q_{Eq}}$ does not really depend on the value of the Equivalent Flow Rate, so it will remain as KC_{E1} .

Therefore, the Equivalent Flow Rate allows the calculation of the Change Gradients and the Energy Cost per meter pumped for a variable discharge regime. The comparison between the two of these two terms allows to obtain the optimum diameter of the pipeline, following the same reasoning that was previously described: Diameter ϕ_i will be optimum (e.g. it should not be passed onto the next diameter of the series ϕ_{i+1}) when:

$$GC_{\phi_i \rightarrow \phi_{i+1}}^{q_{Eq}} > C_{E1} \rightarrow GC_{\phi_i \rightarrow \phi_{i+1}}^{q_{Eq}} > KC_{E1} \cdot \frac{1}{\mu_{B_{Eq}}} \rightarrow \mu_{B_{Eq}} > \frac{KC_{E1}}{GC_{\phi_i \rightarrow \phi_{i+1}}^{q_{Eq}}} \rightarrow \text{Select } \phi_i.$$

As a summary, the method is represented in **figure 3.10**. and it consists of the following steps:

Figure 3.10. Pipe Sizing Methodology using the Equivalent Flow Rate Method.

1. Equivalent Flow Rate is calculated:

$$q_{Eq} = \sqrt{\frac{\sum_1^n V_k q_k^2}{V}}$$

2. With q_{Eq} , the Change Gradients are calculated for each increase in diameter (starting with the smaller in a commercial list and passing onto the very next one).

$$GC_{\phi_i \rightarrow \phi_{i+1}}^{q_{Eq}} = \frac{\pi^2}{n^2 2^{20/3}} \frac{(p_j - p_i)}{\left(\frac{1}{\phi_i^{16/3}} - \frac{1}{\phi_{i+1}^{16/3}} \right)} \frac{1}{q_{Eq}^2}$$

3. Calculate the estimated optimum pump efficiency μ_B using the mathematical adjustment for the average (preferable to the maximum, in order to be conservative) values of μ_B :

$$\mu_B^{Average} = 0.1286 \ln (2.047 \ln q - 1,7951) + 0.547$$

4. Calculate the Annual Cost of energy per meter of head pumped c_{E1} :

$$c_{E1} = 9,81 \frac{V}{3600} \frac{1}{\mu_B} \frac{1}{\mu_M} p$$

5. Calculate the Accumulated Energy Cost per meter of head pumped C_{E1} :

$$C_{E1} = f_A \cdot c_{E1}$$

Being f_A :

$$f_A = \frac{(1+i)^{n_U} - 1}{(1+i)^{n_U} i} \cdot \frac{1}{(1+i)^{n_C}}$$

6. Compare C_{E1} with the Change Gradients $GC_{\emptyset_i \rightarrow \emptyset_{i+1}}^{q_{Eq}}$ and choose the pipe diameter $\emptyset$. The process is:

- If $GC_{\emptyset_i \rightarrow \emptyset_{i+1}}^q < C_{E1}$ → Diameter $\emptyset_{i+1}$ is preferable to $\emptyset_i$ since the cost of reducing 1m the head loss by passing from $\emptyset_{i+1}$ to $\emptyset_i$ is cheaper than the cost of pumping that additional meter.
- If $GC_{\emptyset_i \rightarrow \emptyset_{i+1}}^q > C_{E1}$ → Diameter $\emptyset_i$ is preferable to $\emptyset_{i+1}$ since the cost of reducing 1m the head loss by passing from $\emptyset_{i+1}$ to $\emptyset_i$ is more expensive than the cost of pumping that additional meter.

To sum up, the optimization procedure proposed using the Equivalent Flow Rate is based on the two following premises, which have been demonstrated during the development of the work itself:

- The optimum diameter of the water drive depends only on the pump performance.
- The cost of the full water drive (pipes and pumps) depends only on the pipe diameter.

Therefore, the optimum design of a water drive depends on one only variable, and that is the flow rate. If, as usual, the flow rate is variable, the Equivalent Flow Rate can be used, whose formulation is an original innovation of this work.

3.2.2. Optimization procedure using the Equivalent Volume Method

The Equivalent Volume has been deduced to calculate the Change Gradients when the discharge regime is variable. As a summary, the process is:

1. Choose any value for a virtual constant flow rate q_m . It is recommended that $q_m = 1 \text{ m}^3/\text{s}$. All of the following steps will be shown for this value.
2. For the chosen q_m calculate the Change Gradients series, correspondent to the change of one pipe diameter from a commercial catalogue to the immediate bigger one:

$$GC_{\phi_i \rightarrow \phi_{i+1}}^{q=1} = \frac{\pi^2}{n^2 2^{20/3}} \frac{(p_j - p_i)}{\left(\frac{1}{\phi_i^{16/3}} - \frac{1}{\phi_j^{16/3}} \right)}.$$

3. Calculate the Equivalent Volume for q_m :

$$V_{Eq}^{q_m} = \frac{\sum V_k q_k^2}{q_m^2} \rightarrow V_{Eq}^{q_m=1 \text{ m}^3/\text{s}} = \sum V_k q_k^2.$$

4. Calculate KC_{E1} for $V_{Eq}^{q_m}$:

$$KC_{E1} = \frac{9,81 f_A p}{3600 \mu_M} V_{Eq}^{q_m}.$$

5. Calculate the required pump efficiency needed to each diameter change:

$$\mu_B = \frac{KC_{E1}}{GC_{\phi_i \rightarrow \phi_{i+1}}^{q_{Eq}}}.$$

6. At this point, the same reasoning as for the constant discharge situation is followed: When the pump efficiency reaches a value that could easily be found in the market select diameter ϕ_i . As a reference value, use the results obtained after the pump efficiency analysis:

$$\mu_B^{\text{Average}} = 0.1286 \ln (2.047 \ln q - 1.7951) + 0.5471$$

$$\mu_B^{\text{Maximum}} = 0.0576 \ln (2.047 \ln q - 1.7951) + 0.741$$

being q the circulating flow rate in litres per second (l/s).

For those cases of uncertainty, it is preferable to select a bigger diameter ϕ_{i+1} . The initial investment will be bigger but the risk of additional cost in case a higher energy price will be diminished.

IV. Case Study

4.1. Description

The described methodology has been applied on a case study. This fictional case study is a realistic representation of a branched water supply system of what an agricultural irrigation can be.

The aim of this case study is to determine the diameters of the pipes that conform the representative hydraulic drive. The case study will present a variable water demand, and in this document Granados' System will be carried out using the new concept of the Equivalent Flow Rate and the results of the Pump Analysis to predict the value of the optimum pump efficiency that can be expected.

The topography of the network is schematically represented in **figure 4.1**. It consists of 12 paths, 13 nodes and 5 different agricultural Sectors (S_A , S_B , S_C , S_D and S_E) that will differ in demand distribution.

Figure 4.1. Schematic representation of the hydraulic drive to design.

The dividing nodes have been chosen to mark those places where:

- ▶ There is a variation of the flow rate.
- ▶ There is a variation of the change gradients. It may be due to a change on the rugosity, or and increase on the price of installation due to construction causes. In this way, for Path 1 it has been assumed that since it will have to resist water hammer, it will increase the construction cost by 20%. For the sake of including different conditions, Path 11 has been supposed to cross over land rich in sulphates, which means the pipe will need some sort of protection that will increase the cost by 20% too.
- ▶ The energy line might be compromised due to the topography.
- ▶ It is an initial of an ending not.

The heights of the nots already take into consideration a safety pressure margin that was fixed as one metre of security.

As a result, the network to be designed is a branched water supply system, with the pumping station located at height 610, and that has to overcome a geometrical height difference of up to almost 70 m up to the highest not at the end of path 2 (which is located at the height of 676 m), as well as the head losses of the circulation of the water. The facility has a total length of 38 km of pipes, and the furthest points from the pump station is located at 16.5 km. The average length of the paths 3.2 km long. The exact values are shown in the following table 4.1.

Geometrical characteristics of the case study network			
Path	Initial not height	Ending not height	Length of the path
	m	m	m
1	610	671	1500
2	671	676	2000
3	676	650	500
4	650	670	4500
5	670	675	3000
6	670	655	4500
7	650	630	3000
8	630	622	3000
9	622	590	3000
10	590	595	3500
11	630	640	4000
12	640	625	5500

Table 4.1.

4.2. Water demand

As previously indicated, the case study will be designed using a variable water demand as input. The demand distribution has been designed according to the values reported by *Confederación Hidrográfica del Segura* as the seasonal provisions of the different crops variety held in the Segura watershed, Murcia Region mainly (also Andalusia, Castilla La Mancha, Comunidad Valenciana), Spain, South East.

Each of the sectors represents a different type of plantation and they require different seasonal demands depending on both the kind of crop and the surface of the plantation. The tables containing the calculations of the flow rates can be found in *Annex: Case Study*. As a result, the values of the demand flow rates are shown in the following figure 4.2.

Figure 4.2. Demand distribution for each of the plantations in Sectors A,B,C,D and E. In blue, the aggregated demand in total that would flow through the pumping station.

4.2.1. Calculation of the Equivalent Flow Rate

4.2.1.1. Branch strategy

The Equivalent Flow rate needs to be calculated for every single one of the branches. The following figure 4.3. represents the calculation process. It is discussed underneath.

Figure 4.3. Equivalent flow rate in branched network.

The procedure would consist on:

1. Calculate the demand at the ending nodes. In the graphical representation of **figure 4.3.** above, this would be q_A , q_B and q_C . They might be variable over time, as shown in the representative graphs.
2. Calculate the aggregated demand on the preceding branches. For Path 2, the aggregated demand would be obtained by summing q_A and q_B , and same happens to path 1.
3. Now that every single path has a demand variable demand distribution, calculate the Equivalent Flow Rate for each of the paths. In this way, q_{Eq1} , q_{Eq12} , q_{Eq3} , q_{Eq4} and q_{Eq5} are obtained.
4. Apply Granados' System using the constant Equivalent Flow Rates.

As it may be deduced, the sum of q_{Eq2} and q_{Eq5} is not necessarily q_{Eq1} , (and same for other branches). This variations in the conservation of flow rates summations could be interpreted as a symbolic extraction of injection of water.

This may be surprising at first, but the non-conservation of flow rates is used not only with the Equivalent Flow Method, but also with many other design criteria. For instance, when networks are designed using the maximum flow rate q_{max} that would flow through a path, the summary of maximum flow rates does not necessarily result in q_{max} of the previous branch. This can be seen above in **figure 4.3.** for path 2, 3 and 4: $q_{max\ 3} + q_{max\ 4} \neq q_{max\ 2}$.

It is necessary that the design flow rates are the Equivalent Flow Rates of each own path, and not the sum of the Equivalent Flow Rates of the ending nodes. If q_{Eq} of the ending nodes were to be summed, the resulting flow rates would not have been calculated to mean the same cost of the variable regime, but to only meet a sum.

Calculation

The equivalent flow rate is calculated for each of the demand distributions corresponding each path. Each of the sectors has a different water demand. Nevertheless, for the non-ending paths the aggregated water demand has to be calculated. The values of the water demand are shown underneath in the next **tables 4.2.1. - 4.2.6.**

Water Demand of Sector 1					
Area	500	ha	Annual Volume	3,120,000	m ³ /year
Month	% of the Total Annual Volume	Water Provision	V _k	q _k	v _k *q _k ²
k		m ³ /ha	m ³	m ³ /s	m ¹² /s ²
January	4.4%	275	137,280	0.051	360.64
February	7.5%	468	234,000	0.097	2,189.29
March	5.8%	362	180,960	0.068	826.03
April	8.4%	524	262,080	0.101	2,679.36
May	8.2%	512	255,840	0.096	2,334.29
June	14.6%	911	455,520	0.176	14,068.65
July	15.8%	986	492,960	0.184	16,698.76
August	14.2%	886	443,040	0.165	12,122.10
September	9.2%	574	287,040	0.111	3,520.12
October	4.0%	250	124,800	0.047	270.95
November	4.0%	250	124,800	0.048	289.32
December	3.9%	243	121,680	0.045	251.13
Characteristic Flow Rates			Average	0.10	m ³ /s
			Maximum	0.18	m ³ /s
			Equivalent	0.13	m ³ /s

Table 4.2.1.

Water Demand of Sector 2					
Area	200	ha	Annual Volume	1,248,000	m ³ /year
Month	% of the Total Annual Volume	Water Provision	V _k	q _k	v _k *q _k ²
k		m ³ /ha	m ³	m ³ /s	m ¹² /s ²
January	8.0%	499	99,840	0.037	138.73
February	7.5%	468	93,600	0.039	140.11
March	5.8%	362	72,384	0.027	52.87
April	8.0%	499	99,840	0.039	148.13
May	13.0%	811	162,240	0.061	595.28
June	10.0%	624	124,800	0.048	289.32
July	15.8%	986	197,184	0.074	1,068.72
August	12.0%	749	149,760	0.056	468.21
September	7.0%	437	87,360	0.034	99.24
October	5.0%	312	62,400	0.023	33.87
November	4.0%	250	49,920	0.019	18.52
December	3.9%	243	48,672	0.018	16.07
Characteristic Flow Rates			Average	0.04	m ³ /s
			Maximum	0.07	m ³ /s
			Equivalent	0.05	m ³ /s

Table 4.2.2.

Water Demand of Sector 3					
Area	600	ha	Annual Volume	3,744,000	m ³ /year
Month	% of the Total Annual Volume	Water Provision	V _k	q _k	v _k *q _k ²
k		m ³ /ha	m ³	m ³ /s	m ¹² /s ²
January	5.0%	312	187,200	0.070	914.46
February	6.0%	374	224,640	0.093	1,936.95
March	8.0%	499	299,520	0.112	3,745.64
April	8.0%	499	299,520	0.116	3,999.52
May	13.0%	811	486,720	0.182	16,072.62
June	14.0%	874	524,160	0.202	21,434.91
July	14.0%	874	524,160	0.196	20,074.32
August	12.0%	749	449,280	0.168	12,641.55
September	10.0%	624	374,400	0.144	7,811.56
October	5.0%	312	187,200	0.070	914.46
November	4.0%	250	149,760	0.058	499.94
December	1.0%	62	37,440	0.014	7.32
Characteristic Flow Rates			Average	0.12	m ³ /s
			Maximum	0.20	m ³ /s
			Equivalent	0.16	m ³ /s

Table 4.2.3.

Water Demand of Sector 4					
Area	280	ha	Annual Volume	1,747,200	m ³ /year
Month	% of the Total Annual Volume	Water Provision	V _k	q _k	v _k *q _k ²
k		m ³ /ha	m ³	m ³ /s	m ¹² /s ²
January	1.0%	62	17,472	0.007	0.74
February	3.0%	187	52,416	0.022	24.61
March	5.0%	312	87,360	0.033	92.94
April	7.0%	437	122,304	0.047	272.30
May	11.0%	686	192,192	0.072	989.59
June	16.0%	998	279,552	0.108	3,251.75
July	14.0%	874	244,608	0.091	2,040.15
August	12.0%	749	209,664	0.078	1,284.76
September	11.0%	686	192,192	0.074	1,056.66
October	10.0%	624	174,720	0.065	743.49
November	7.0%	437	122,304	0.047	272.30
December	3.0%	187	52,416	0.020	20.07
Characteristic Flow Rates			Average	0.06	m ³ /s
			Maximum	0.11	m ³ /s
			Equivalent	0.08	m ³ /s

Table 4.2.4.

Water Demand of Sector 5					
Area	430	ha	Annual Volume	2,683,200	m ³ /year
Month	% of the Total Annual Volume	Water Provision	V _k	q _k	v _k *q _k ²
k		m ³ /ha	m ³	m ³ /s	m ¹² /s ²
January	4.0%	250	107,328	0.040	172.34
February	7.0%	437	187,824	0.078	1,132.16
March	9.0%	562	241,488	0.090	1,963.07
April	9.0%	562	241,488	0.093	2,096.12
May	10.0%	624	268,320	0.100	2,692.83
June	11.0%	686	295,152	0.114	3,827.08
July	12.0%	749	321,984	0.120	4,653.20
August	11.0%	686	295,152	0.110	3,584.15
September	9.0%	562	241,488	0.093	2,096.12
October	7.0%	437	187,824	0.070	923.64
November	6.0%	374	160,992	0.062	621.07
December	5.0%	312	134,160	0.050	336.60
Characteristic Flow Rates			Average	0.09	m ³ /s
			Maximum	0.12	m ³ /s
			Equivalent	0.09	m ³ /s

Table 4.2.5.

The values of the demand on paths where the flow rate is a combination of the demands of the sectors are shown in the following tables 4.2.6 - 4.2.9.

S1+S2			
Annual Volume	4,368,000	m ³ /year	
Month	V _k	q _k	v _k *q _k ²
k	m ³	m ³ /s	m ¹² /s ²
January	237,120	0.09	1,858.5
February	327,600	0.14	6,007.4
March	253,344	0.09	2,266.6
April	361,920	0.14	7,056.1
May	418,080	0.16	10,186.6
June	580,320	0.22	29,089.3
July	690,144	0.26	45,821.4
August	592,800	0.22	29,038.5
September	374,400	0.14	7,811.6
October	187,200	0.07	914.5
November	174,720	0.07	793.9
December	170,352	0.06	689.1
Characteristic Flow Rates		Average	0.14
		Maximum	0.26
		Equivalent	0.18

Table 4.2.6.

S4+S3			
Annual Volume	5,491,200	m ³ /year	
Month	V_k	q_k	v_k*q_k²
k	m ³	m ³ /s	m ¹² /s ²
January	204,672	0.08	1,195.2
February	277,056	0.11	3,633.8
March	386,880	0.14	8,071.9
April	421,824	0.16	11,171.8
May	678,912	0.25	43,620.4
June	803,712	0.31	77,273.6
July	768,768	0.29	63,333.7
August	658,944	0.25	39,883.6
September	566,592	0.22	27,073.3
October	361,920	0.14	6,608.3
November	272,064	0.10	2,997.4
December	89,856	0.03	101.1
Characteristic Flow Rates	Average	0.17	m ³ /s
	Maximum	0.31	m ³ /s
	Equivalent	0.23	m ³ /s

Table 4.2.7

S5+S4+S3			
Annual Volume	8,174,400	m ³ /year	
Month	V_k	q_k	v_k*q_k²
k	m ³	m ³ /s	m ¹² /s ²
January	312,000	0.12	4,233.6
February	464,880	0.19	17,166.4
March	628,368	0.23	34,585.3
April	663,312	0.26	43,439.4
May	947,232	0.35	118,472.7
June	1,098,864	0.42	197,497.6
July	1,090,752	0.41	180,895.3
August	954,096	0.36	121,066.9
September	808,080	0.31	78,540.4
October	549,744	0.21	23,159.6
November	433,056	0.17	12,088.2
December	224,016	0.08	1,567.1
Characteristic Flow Rates	Average	0.26	m ³ /s
	Maximum	0.42	m ³ /s
	Equivalent	0.32	m ³ /s

Table 4.2.8

S1+S2+S3+S4+S5			
Annual Volume	12,542,400	m ³ /year	
Month k	V _k m ³	q _k m ³ /s	v _k *q _k ² m ¹² /s ²
January	549,120	0.21	23,080.8
February	792,480	0.33	85,039.6
March	881,712	0.33	95,549.7
April	1,025,232	0.40	160,397.1
May	1,365,312	0.51	354,768.3
June	1,679,184	0.65	704,733.4
July	1,780,896	0.66	787,344.5
August	1,546,896	0.58	515,979.5
September	1,182,480	0.46	246,099.9
October	736,944	0.28	55,789.5
November	607,776	0.23	33,416.5
December	394,368	0.15	8,549.8
Characteristic Flow Rates	Average	0.40	m ³ /s
	Maximum	0.66	m ³ /s
	Equivalent	0.49	m ³ /s

Table 4.2.9

The following **table 4.3.** shows the values of the seasonal flow rates required, as well design flow rates q_{Eq} . The average and the maximum value of the flow rates are also shown, just for information.

Plantation Sector									
	A	B	C	D	E	A+B	C+D	C+D+E	A+B+C +D+E
Path									
	5	6	9, 10	None	11, 12	4	8	7	1, 2, 3
Month k	Flow rate, q _k								
	m ³ /s	m ³ /s	m ³ /s	m ³ /s	m ³ /s	m ³ /s	m ³ /s	m ³ /s	m ³ /s
January	0.05	0.04	0.07	0.01	0.04	0.09	0.08	0.12	0.21
February	0.10	0.04	0.09	0.02	0.08	0.14	0.11	0.19	0.33
March	0.07	0.03	0.11	0.03	0.09	0.09	0.14	0.23	0.33
April	0.10	0.04	0.12	0.05	0.09	0.14	0.16	0.26	0.40
May	0.10	0.06	0.18	0.07	0.10	0.16	0.25	0.35	0.51
June	0.18	0.05	0.20	0.11	0.11	0.22	0.31	0.42	0.65
July	0.18	0.07	0.20	0.09	0.12	0.26	0.29	0.41	0.66
August	0.17	0.06	0.17	0.08	0.11	0.22	0.25	0.36	0.58
September	0.11	0.03	0.14	0.07	0.09	0.14	0.22	0.31	0.46
October	0.05	0.02	0.07	0.07	0.07	0.07	0.14	0.21	0.28

November	0.05	0.02	0.06	0.05	0.06	0.07	0.10	0.17	0.23
December	0.05	0.02	0.01	0.02	0.05	0.06	0.03	0.08	0.15
Average	0.10	0.04	0.12	0.06	0.09	0.14	0.17	0.26	0.40
Maximum	0.18	0.07	0.20	0.11	0.12	0.26	0.31	0.42	0.66
Design Value: Equivalent Flow Rate									
Equivalent	0.13	0.05	0.16	0.08	0.09	0.18	0.23	0.32	0.49

Table 4.3.

4.2.2. Initial solution

As a summary, the initial characteristics of the network are compiled in the following table 4.4.

Network characteristics					
Path	Previous Path	Extra costs Sulphates and water hammer	Length m	Flow Rates m ³ /s	Ending Heights m
1	0	1.20	1500	0.49	671
2	1	1.00	2000	0.49	676
3	2	1.00	500	0.49	650
4	3	1.00	4500	0.18	670
5	4	1.00	3000	0.13	675
6	4	1.00	4500	0.05	655
7	3	1.00	3000	0.32	630
8	7	1.00	3000	0.23	622
9	8	1.00	3000	0.16	590
10	9	1.00	3500	0.16	595
11	7	1.20	4000	0.09	640
12	11	1.0	5500	0.09	625

Table 4.4.

With this assignation of the design flow rates, Granados' System can be applied. For the application of the Granados' System, a commercial collection of pipe diameters along with their prices has to be selected. Table 4.5. shows the available diameters and the respective prices per meter chosen for this case study.

Granados' System requires an initial solution that will then be modify and optimise onto the final solution. This initial solution ensures that the system meets the requirements of not surpassing a maximum velocity. In this case, the maximum velocity is defined as $v_{\max} [\text{m/s}] = 2 + \emptyset$, expressing the commercial diameter $\emptyset$ in metres. This is only an example of how the maximum velocity can

be defined. Once the maximum velocity is defined, the maximum flow rate that can circulate through the specific tube can be obtained, as show in **table 4.5**. With this, each path of the network can be assigned an initial diameter, ensuring that the maximum velocity will never be surpassed. From this point on, the optimisation will only increase the pipe diameter.

Pipe catalogue and maximum flow rate			
ϕ	Price	$v_{\max}$	$q_{\max}$
mm	€/1000 m	m/s	m ³ /s
1300	1,760,872	3.3	4.4
1200	1,369,891	3.2	3.6
1100	1,048,281	3.1	2.9
1000	786,380	3.0	2.4
900	630,871	2.9	1.8
800	545,380	2.8	1.4
700	416,600	2.7	1.0
600	347,435	2.6	0.7
500	281,487	2.5	0.5
450	249,719	2.5	0.4
400	218,755	2.4	0.3
350	169,241	2.4	0.2
300	127,178	2.3	0.2
250	96,133	2.3	0.1
200	76,001	2.2	0.1

Table 4.5.

The initial solution consists on determining what the pumping head has to be in order to have enough energy to reach all of the nodes with the initial diameter, considering all the head losses that will occur along the way. For that, the procedure consisted on assuming there is no pump in the first node, but the water still circulates for every single path at the Equivalent Flow Rate losing energy. In such way, you end up with the network full of negative pressures at the nodes. The node with the maximum negative pressure will determine the initial pumping head required for the initial solution. In order to calculate the head loss, Manning's Formula is used. For the case study, a cast iron pipe was supposed, which has a Manning rugosity of 0.012. The results of the initial solution are shown in the following **table 4.6**.

Initial Solution					
Pumping height		610	Pumping height		884.75
Path	φ min	Initial height	Final Height	Pressure margin	Pressure margin
	mm	m	m	m	m
1	600	610.00	601.70	-69.30	205.45
2	600	601.70	590.63	-85.37	189.38
3	600	590.63	587.87	-62.13	212.61
4	350	587.87	529.47	-140.53	134.22
5	300	529.47	480.75	-194.25	80.50
6	200	529.47	441.82	-213.18	61.57
7	450	587.87	555.83	-74.17	200.58
8	400	555.83	525.25	-96.75	177.99
9	300	525.25	459.50	-130.50	144.24
10	300	459.50	382.79	-212.21	62.53
11	250	555.83	469.27	-170.73	104.02
12	250	469.27	350.25	-274.75	0.00
HP min				-274.75	

Table 4.6

4.2.3. Pipe sizing procedure

4.2.3.1. Change Gradients calculation

All calculations of the Change Gradients are shown in the following **tables 4.7.1-4.7.3**. They begin by calculating the head losses and the pipe prices (taking into account the extra costs due to the water hammer and the sulphates in path 1 and 11) for each of the paths, considering all the possible commercial diameters, so that, even though some of the values might not be used, the program works easier using the functions Index and Position later on in the iterations.

For the calculation of the Change Gradients, firstly, the possible head losses along each path with all different commercial diameters were calculated, so that it was easy to program it with the Index Function and Position. These values are shown in the following **table 4.7.1**.

Δh: Compilation of Head Losses in each Path												
φ	1	2	3	4	5	6	7	8	9	10	11	12
1300	0.1	0.2	0.0	0.1	0.0	0.0	0.1	0.1	0.0	0.0	0.0	0.0
1200	0.2	0.3	0.1	0.1	0.0	0.0	0.2	0.1	0.0	0.0	0.0	0.0
1100	0.3	0.4	0.1	0.1	0.0	0.0	0.3	0.1	0.1	0.1	0.0	0.0
1000	0.5	0.7	0.2	0.2	0.1	0.0	0.5	0.2	0.1	0.1	0.1	0.1
900	1.0	1.3	0.3	0.4	0.1	0.0	0.8	0.4	0.2	0.2	0.1	0.1
800	1.8	2.4	0.6	0.7	0.3	0.1	1.5	0.8	0.4	0.4	0.2	0.2
700	3.6	4.9	1.2	1.4	0.5	0.1	3.0	1.5	0.7	0.8	0.4	0.5
600	8.3	11.1	2.8	3.3	1.2	0.3	6.9	3.5	1.6	1.9	0.8	1.1
500	21.9	29.3	7.3	8.7	3.2	0.7	18.3	9.3	4.3	5.0	2.1	3.0
450	38.5	51.3	12.8	15.3	5.6	1.2	32.0	16.3	7.6	8.8	3.8	5.2
400	72.1	96.2	24.0	28.6	10.5	2.2	60.0	30.6	14.2	16.5	7.1	9.7
350	147.1	196.1	49.0	58.4	21.4	4.4	122.4	62.3	28.9	33.7	14.4	19.8
300	334.6	446.2	111.5	132.9	48.7	10.1	278.5	141.9	65.8	76.7	32.7	45.0
250	884.9	1,179.8	295.0	351.3	128.8	26.7	736.3	375.1	173.9	202.8	86.6	119.0
200	2,908.9	3,878.5	969.6	1,154.9	423.5	87.7	2,420.6	1,233.1	571.5	666.8	284.6	391.3
Q	0.5	0.5	0.5	0.2	0.1	0.0	0.3	0.2	0.2	0.2	0.1	0.1
Length	1500	2000	500	4500	3000	4500	3000	3000	3000	3500	4000	5500

Table 4.7.1..

The same reasoning is applied for the pipe prices. The results are shown in **table 4.7.2.**

Price of the pipe in each Path for all possible diameters												
Path	1	2	3	4	5	6	7	8	9	10	11	12
1300	3,169 ,570	3,521 ,744	880,4 36	7,923 ,924	5,282 ,616	7,923 ,924	5,282 ,616	5,282 ,616	5,282 ,616	6,163 ,052	8,452 ,186	9,684 ,796
1200	2,465 ,804	2,739 ,782	684,9 46	6,164 ,510	4,109 ,673	6,164 ,510	4,109 ,673	4,109 ,673	4,109 ,673	4,794 ,619	6,575 ,477	7,534 ,401
1100	1,886 ,906	2,096 ,562	524,1 41	4,717 265	3,144 ,843	4,717 265	3,144 ,843	3,144 ,843	3,144 ,843	3,668 ,984	5,031 ,749	5,765 ,546
1000	1,415 484	1,572 760	393,1 90	3,538 ,710	2,359 ,140	3,538 ,710	2,359 ,140	2,359 ,140	2,359 ,140	2,752 ,330	3,774 ,624	4,325 ,090
900	1,135 568	1,261 742	315,4 36	2,838 ,920	1,892 ,613	2,838 ,920	1,892 ,613	1,892 ,613	1,892 ,613	2,208 ,049	3,028 ,181	3,469 ,791
800	981,6 84	1,090 ,760	272,6 90	2,454 ,210	1,636 ,140	2,454 ,210	1,636 ,140	1,636 ,140	1,636 ,140	1,908 ,830	2,617 824	2,999 ,590
700	749,8 80	833,2 00	208,3 00	1,874 ,700	1,249 ,800	1,874 ,700	1,249 ,800	1,249 ,800	1,249 ,800	1,458 ,100	1,999 ,680	2,291 300
600	625,3 83	694,8 70	173,7 18	1,563 ,458	1,042 ,305	1,563 ,458	1,042 ,305	1,042 ,305	1,042 ,305	1,216 023	1,667 688	1,910 893

500	506,6 77	562,9 74	140,7 44	1,266 ,692	844,4 61	1,266 ,692	844,4 61	844,4 61	844,4 61	985,2 05	1,351, 138	1,548 ,179
450	449,4 94	499,4 38	124,8 60	1,123, 736	749,1 57	1,123, 736	749,1 57	749,1 57	749,1 57	874,0 17	1,198, 651	1,373, 455
400	393,7 59	437,5 10	109,3 78	984,3 98	656,2 65	984,3 98	656,2 65	656,2 65	656,2 65	765,6 43	1,050 ,024	1,203, 153
350	304,6 34	338,4 82	84,62 1	761,5 85	507,7 23	761,5 85	507,7 23	507,7 23	507,7 23	592,3 44	812,3 57	930,8 26
300	228,9 20	254,3 56	63,58 9	572,3 01	381,5 34	572,3 01	381,5 34	381,5 34	381,5 34	445,1 23	610,4 54	699,4 79
250	173,0 39	192,2 66	48,06 7	432,5 99	288,3 99	432,5 99	288,3 99	288,3 99	288,3 99	336,4 66	461,4 38	528,7 32
200	136,8 02	152,0 02	38,00 1	342,0 05	228,0 03	342,0 05	228,0 03	228,0 03	228,0 03	266,0 04	364,8 05	418,0 06
Q	0.495	0.495	0.495	0.180	0.134	0.050	0.319	0.228	0.155	0.155	0.095	0.095
Length	1500	2000	500	4500	3000	4500	3000	3000	3000	3500	4000	5500
Extra Cost	1.2	1	1	1	1	1	1	1	1	1	1.2	1

Table 4.7.2.

Once all the head losses and pipe prices have been calculated, the Changes Gradients are directly obtained. These can be seen in **table 4.7.3**.

Change Gradients in each Path												
φ	1	2	3	4	5	6	7	8	9	10	11	12
1300												
1200	9,838, 516	8,198, 763	8,198, 763	61,949 ,214	112,61 8,212	816,24 6,854	19,704 ,837	38,68 0,224	83,45 4,247	83,45 4,247	268,19 9,624	223,4 99,687
1100	4,761, 936	3,968, 280	3,968, 280	29,98 4,012	54,50 8,292	395,07 1,284	9,537, 330	18,721, 598	40,39 2,654	40,39 2,654	129,81 1,183	108,17 5,986
1000	2,173, 212	1,811,0 10	1,811,0 10	13,683 ,852	24,87 6,038	180,29 9,325	4,352, 567	8,544, 006	18,434 ,061	18,434 ,061	59,24 2,141	49,36 8,451
900	681,94 3	568,2 86	568,2 86	4,293, 926	7,805, 979	56,577 ,043	1,365, 814	2,681, 067	5,784, 518	5,784, 518	18,589 ,893	15,491, 578
800	184,35 9	153,63 2	153,63 2	1,160,8 32	2,110,2 90	15,295 ,199	369,2 38	724,8 07	1,563, 803	1,563, 803	5,025, 645	4,188, 037
700	124,74 3	103,95 3	103,95 3	785,4 58	1,427, 893	10,349 ,243	249,8 39	490,4 29	1,058,1 21	1,058,1 21	3,400, 519	2,833, 766
600	26,761	22,301	22,301	168,50 2	306,3 22	2,220, 194	53,597	105,21 0	226,9 96	226,9 96	729,50 4	607,92 0
500	8,698	7,249	7,249	54,770	99,56 6	721,64 7	17,421	34,197	73,782	73,782	237,11 6	197,59 7
450	3,455	2,879	2,879	21,757	39,55 2	286,6 69	6,920	13,585	29,30 9	29,30 9	94,193	78,49 4
400	1,656	1,380	1,380	10,428	18,958	137,40 4	3,317	6,511	14,048	14,048	45,148	37,623
350	1,190	991	991	7,490	13,617	98,69 5	2,383	4,677	10,091	10,091	32,42 9	27,02 4
300	404	336	336	2,542	4,621	33,49 0	808	1,587	3,424	3,424	11,004	9,170
250	102	85	85	639	1,163	8,426	203	399	861	861	2,769	2,307
200	18	15	15	113	205	1,485	36	70	152	152	488	407

Table 4.7.3.

4.2.3.2. Optimum pump efficiency calculation

Using the expression obtained in the Pump Analysis, the average Optimum pump efficiency is calculated, using the Equivalent Flow Rate that would circulate through the pump station, which is the same as the one for Path 1.

$$\text{Pump } q_{Eq} = \text{Path}_1 q_{Eq} = 490.8 \text{ [l/s]} ,$$

$$\mu_B^{Av} = 0.1286 \ln (2.047 \ln(490.8) - 1,7951) + 0.5471 = 0.85$$

Figure 4.4. Optimum pump efficiency expected for the Case Study using the Equivalent Flow Rate through path 1.

4.2.3.3. Energy cost calculation

The cost of electrical energy for the operation of pumping station has to be calculated considering the real total volume of water to be pumped. This means that it needs to be obtained throughout the actual variable regime, and not using the equivalent flow rate. As shown above, the pump efficiency has been calculated using the expression obtained from the pump analysis. The estimated efficiency of the engine has been fixed to 0.93, and the discount factor will account for a lifespan of the utility of 20 years, and 1 for the construction, based on realistic values. The total accumulated cost of electrical energy per meter of pumping head finally results in 6,1265.3 €/m. This is the cost to be compared to the Change Gradients.

Total Cost per meter of pumping head		
Total C1	6,1265.3	€/m
Annual c1	5,161.9	€/m*year
Volume	12,542,400	m ³ /year
Energy price: p _e	0.12	€/kWh
$\mu_B \cdot \mu_M$	0.79	%
Pumping efficiency		
Average μ_B	0.85	%
Maximum μ_B	0.88	%
μ_M	0.93	%
Discount factor		
f _a	11.87	
i: Interest rate	0.05	
Life span of the utility	20	years
Construction years	1	years

Table 4.8.

4.2.3.4. Optimisation

Granados' System is based on comparing the Change Gradients (cost of increasing the pipe and reducing the head loss) to the Energy Cost (pumping the head losses instead of reducing them). Granados has explained it in depth along his career path, and it has been used for the design of hydraulic systems in Spain. None the less, a brief summary of the steps is explained as follows. These are the steps have been followed to resolve this case study.

The general idea of the procedure consists on selecting a path (or several paths) and proposing the increase of its diameter. If CG of the path < C_{E1}, the diameter is increased, the pumping head is reduced, and another path is selected for a new iteration. If CG of the path > C_{E1}, no diameter is increased, and the designing process has been completed.

The way to select the path/s to change in each iteration follows these steps:

1. Locate those nodes with no pressure margin. From those points, find the course up to the pumping station: The critical course. Those paths in the critical course are the possible options to optimise. They are the change alternatives.

In case there are more than one not with no pressure margin, select only one path from each critical course to conform a change alternative. More than one path may be selected.

2. Calculate the CG of the change alternatives. If there are several paths involved in the alternative, sum up their respective CGs. The selection among the change alternatives, corresponds to that of the minimal Change Gradient.
3. If CG (or $\sum CG < CE1$), diameter/s may be increased. Bear in mind that the increase of the diameter of the selected path/s is conditioned by the pressure margin of the rests of the nots. This may be seen in the following figure 4.5.

Figure 4.5. (a), represents the initial solution at the beginning of an iteration. The change alternatives would be: **figure 4.5. (b)** changing ϕ_a into ϕ_A , or Figure (c) changing ϕ_c into ϕ_C . However, if the change of ϕ_a into ϕ_A compromises the pressure margin of another not, like in **figure 4.5. (d)**, the path cannot be fully amplified, but partially instead, as seen in **figure 4.5. (e)**.

If the resulting situation after the iteration was left as in **figure 4.5. (e)** for the next iteration, the change alternatives will be: Changing ϕ_c , or changing both ϕ_a and ϕ_b at the same time. Keep in mind that, as **figure 4.5. (f)** shows, the mixed path will only increase the size of the smallest diameter, and the pumping head reduction will only be diminished in what was left from the previous iteration.

All iterations can be seen in *Annex: Case Study*. It took 19 iterations to find the optimal solution. On the 20th iteration, $CE1$ was bigger than the CG, meaning that no more diameters could be engrossed, and that, therefore, the optimum solution had been obtained.

Figure 4.5. Iteration rules in the pipe optimization.

4.3. Results of the Case Study and Discussion

4.3.1. Optimised solution

The final design values are shown in the following **table 4.9**.

Optimal Solution						
Path	Initial Solution. Minimum diameter	Energy height at the end not	Pressure margin	Optimal diameters	Length of the full path	Length of the bigger pipe
		m	m	mm	m	m
1	600	671	24.4	700	1500	
2	600	676	14.5	700	2000	
3	600	650	39.3	700	500	
4	350	670	10.6	500	4500	
5	300	675	0.0	450	3000	
6	200	655	0.0	300 250	4500	287.3
7	450	630	47.5	600 500	3000	1706.8
8	400	622	35.6	450 400	3000	2245.7
9	300	590	38.7	350	3000	
10	300	595	0.0	350	3500	
11	250	640	4.8	300	4000	
12	250	625	0.0	350	5500	
Manometric height		696.6	m			
Pumping head		86.6	m			

Table 4.9.

The second column shows the diameters calculated as the initial solution. They were calculated in order to ensure that the maximum velocity was never surpassed. From that base on, the diameters could only be increased. Nevertheless, they were not calculated as the most economical solution. For that, the respective iterations were carried out to find the optimum diameter combinations, which are shown in the 6th and 7th column of the previous table.

As shown, all of the paths increased their diameter in, at least, one step into the next commercial diameter of the manufacture's series. The biggest transformation was for path 4 and 5, which climbed up to 3 bigger diameters. It can also be seen that some of the paths have two different diameters. It is the case of path 6, 7 and 8. This means that the paths present both different diameters. Column number 8 shows the length of those sections of the path with

the bigger diameter. Path 8 is almost fully constituted by the bigger diameter (2245.7 m out of the 3000 m of the full length), which means that the limitation of the pressure margin was very little. Nevertheless, that was not the case for path 6 (287.3 m out of the total 4500 m) which had very little room to be engrossed from the beginning of the iteration. These mixed paths would correspond to the case shown in **figure 4.5. (f)**, where not the full length of the entire path could increase its diameter in order to not compromise the pressure margin of a network's not.

The 4th column shows precisely that pressure margin values. Keep in mind that, as mentioned at the beginning of the description of the case study, the topography of the network already includes one metre of pressure margin for safety reasons. The biggest pressure margin is located in the ending not of path 7, and it rises up to a total 47.5 m of pressure margin. On the other hand, the ending nots of paths 5,6,10 and 12 have no pressure margin. Therefore, the network design is has been tighten around those nots and they have been the limiting nots to optimise the network. In Annex, all calculations are shown, and it may be seen that the ending not at path 12 was the first to have no pressure margin. In fact, it was the determining not for the initial solution. The ending not of path 6 hit the no pressure margin on the 1st iteration, whilst for the ending nots of path 10 and 5, it happened on the 2nd and 4th iteration, respectively.

The pumping head required at the beginning of the design process was 274.74 m. This value is what the initial solution required, using the diameters that strictly required not overcoming the value of the maximum speed. This was a much bigger value than the pumping head obtained from the solution: 86.6 m, which means that the increase of the pipe's diameter has effectively reduced the energy expenses. Nevertheless, it has to be remembered that this is the pumping head that the system would require if the Equivalent Flow Rate was to circulate, which is not the real situation. The real regime would require different and variable flow rates depending on the month, and therefore, the pumping head will vary (but meaning the same electrical cost at the end of the year). The following **figure 4.6.** shows the diverse values to the pumping head required for the variable flow regime. Bear in mind that these are the 'aimed functioning points'; the real functioning points will adapt to the selected pump, once it has been acquired (but then again, the pump will be chosen in order to function as chose as possible to these 'aimed functioning points').

Figure 4.6. Pumping head depending on the flow rate regime.

As it can be seen, the first four months of the year, the required pumping head is slightly smaller (around 70 metres required) than the theoretical one required by the Equivalent Flow Rate, as well as for October, November and December. This makes seven months of very similar conditions to the theoretical constant regime of q_{Eq} . The most demanding month would be June, when it doubles the height of the other months, requiring up to 197.9 m of pumping head. The exact values are exposed in the *Annex: Case Study*.

4.3.2. Comparison with variable demand.

As a verification, the energy cost of the facility working with the optimised diameters shown in the solution has been calculated for both, the variable flow rate regime, and the Equivalent Flow Rate. By definition of the Equivalent Flow Rate, they ought to throw back the same annual cost. **Figure 4.7.** shows the accumulated energy cost by month, depending on the flow rate regime used.

Figure 4.7. Accumulated energy expenses depending on the flow rate regime.

As expected, both regimes, meet at the same point, meaning that the accumulated annual cost of energy is the same. Whilst the Equivalent Flow Rate Regime grows at almost the same rate month by month (small variations can occur due to the different number of days over a month), the variable flow rate regime grows differently in each month. Since the energy cost is, by definition, affected by the pumping head and the volume of water to be pumped, it is expected that those months requiring higher volumes of water and pumping head (like June and July) show the highest inclination in the curve.

The Equivalent Flow Rate has been proved to be an effective method to simplify the design of a complex water supply system, adapting long proved design methods such as the Granados' System to the variability of the water demand, which is becoming an increasingly more frequent requirement. The pump analysis has substantially facilitated the calculation procedure, converting Granados' System into a now a straightforward methodology.

Conclusions

In this chapter, the conclusions of the research are presented. It begins with a brief summary of the thesis, and it is then followed by a resume of the original contributions of this work. Finally, this work will conclude by indicating the future areas of advancement that can be done in the field.

General summary

The aim of this thesis was to propose a new design methodology of water supply systems that would take into consideration the variations in the demand of water, as well as a proper energy assessment of the electrical expenses through the lifespan of the utility. This new design methodology will be used to determine the pipe diameters of a water supply system, whilst it is conceived to minimise all costs derived from both, construction and operation of the facility.

The origin of the method: Granados' System

Granados' System is the initial procedure on which the new methodology is based. The reasoning behind Granados' System is to find which of the following options is cheaper: It could be selecting a small but cheap pipe that will mean high head losses and therefore big energy costs; or on the contrary, it could be selecting a big and expensive pipe that will minimise head losses and reduce energy expenses during the operation life. This intuitive reasoning is what has made Granados' System so successful and used in practice.

Granados' System bases its logic on the concept of the Change Gradient. The Change Gradient, CG, represents the cost of reducing the head loss in the conduction by one metre by building a larger pipe. It symbolises the cost of the second option: choosing the big pipe. In order to find what is cheaper, it needs to be compared to the cost of energy used for pumping one metre of head loss during the full lifespan of the installation, C_{E1} . Starting from the smallest commercial diameter available, both CG and C_{E1} are calculated and compared, and the iterative process begins: If CG is smaller (and therefore, cheaper), it means that it is more economic to increase the pipe diameter onto the next commercial one. On the other hand, when C_{E1} is smaller (and therefore, cheaper), it will mean

that it is more convenient to leave a narrow pipe, and that the optimum design has been found.

Limitations of the Granados' System

However, Granados' System has two limitations:

- ▶ The calculation of the Change Gradients needs a constant design flow rate. Nevertheless, having constant demand pattern rarely happens, and it is way more common to have a variable demand regime.
- ▶ The calculation of the energy cost C_{E1} , requires the knowledge of the pump efficiency. In the absence of a precise definition of this factor, prior to the elaboration of this thesis, the value of the pump efficiency μ_B was guessed by series of additional iterations that would difficult the calculations.

The purpose of this thesis was to find a solution to these two limitations, and then reorganise the procedure, and apply it in a case study. These conform the main goals of the research, which were presented at the introduction chapter:

1. Adapt variable demand patterns in the design methodology.
2. Define the expected values of the pump efficiency.
3. Organise the steps of the design methodology and applicate them in a case study.

The mathematical demonstration of both limitations of Granados' System, and solutions, have been shown in depth in the Methodology chapter.

Overcoming the variable demand regime barrier

In order to overthrow the first of the limitations, variability of the demand, because Granados' System only works for constant flow rates, the idea was to find a theoretical constant flow rate that could be used as the design flow rate, that represented the same costs that the variable regime would have meant, since it is the costs what are being optimised. As a conclusion, the concepts of the Equivalent Flow Rate and Equivalent Volume were elaborated as two possible solutions:

- ▶ The **Equivalent Flow Rate** q_{Eq} is defined as a theoretical flow rate for which, if all the volume V required in a year was pumped at this discharge, the cost of the pumping energy would be the same as the cost of pumping at a variable flow rate regime.
- ▶ If a constant flow rate q_m was chosen to design the pipe line, $V_{Eq}^{q_m}$ is the total **Equivalent Volume** of water that needs to be pumped at that flow

rate q_m to make the final cost of pumping equal to what it would be pumping at a variable regime of flow rates.

By using any of these two concepts, the Change Gradients can be calculated, and the method is adapted to work with variable demand regimes. In the case study, the concept that was used was that of the Equivalent Flow Rate.

Defining the pump efficiency

An in depth analysis of the pump efficiency μ_B was carried out in order to provide a precise estimation of what optimum value of μ_B can be expected. This will break down a loop process of the Granados' System and will help to simplify calculations. The optimum pump efficiency μ_B presents a wide range of possible values that typically goes from 70% up to 90%. This means a variability of up to 20% in the estimation of the cost, which is a substantial difference. The definition of μ_B will allow designers to perform a proper energy assessment, and therefore a better design.

The idea was to determine if it existed any relationship between the pump efficiency and any of the other design variables involved. For this to be done, 226 commercial pumps from the catalogues of several manufactures were selected, and the technical features at the optimum operation point were collected and organised. All of this data collection was plotted, showing significant results:

- ▶ There is a clear relationship between the pump efficiency and the flow rate. The relationship grows very fast for smaller than 200 l/s flow rate, coming from 50% up to almost 83%. Between 500 and 200 l/s the growing rate diminishes, although the curve arrives at 87% pump efficiency by the end of the interval. Above 500 l/s, the relationship is pretty stable and μ_B stabilises around 87%, slightly increasing towards a 90% asymptote.
- ▶ There is no sign of relationship between the pump efficiency and the pumping head.
- ▶ There is a relationship between the pump efficiency and the required power for the optimum operating point. However, this relationship is weaker than the one of the flow rate.
- ▶ Regarding the pump type, split case pumps are the ones that provide the best efficiency for flow rates over 500 l/s.
- ▶ There is a diffuse tendency of μ_B growth as the specific speed increases.

The relationship between the pump efficiency and the flow rate is very clear, enabling a very precise mathematical adjustment. It was made through a double logarithmic curve, which fits very satisfactory ($r^2 > 98\%$). With this empiric

relationship designers can now adequately predict the value of the pump efficiency, and the Granados' System can be simplified, avoiding unnecessary iterations.

Assembling the complete procedure

With all of the above contributions, the method can be assembled. The detailed steps of the procedure are explained in the results section of this thesis, in both, the Optimization procedure using the Equivalent Flow Rate Method section, and the Optimization procedure using the Equivalent Volume Method section. The Equivalent Flow Rate Method is more intuitive and allows an easier physical understanding than the Equivalent Volume. For these reasons, it was selected to illustrate the design of the case study.

Case study

The case study presented in this PhD is a realistic representation of a branched water supply system an agricultural irrigation. The aim is to determine the diameters of the pipes that conform the symbolised hydraulic drive. The case study presents a variable water demand, and as previously said, it was solved by using the Equivalent Flow Rate Method. It fully illustrates the design process of the methodology proposed in this work, in a complex branched network, It consists of one pumping station, 12 paths, 13 nodes and 5 different agricultural Sectors that differ in demand distribution. The process begins by calculating the Equivalent Flow Rates q_{Eq} . Each Branch has a different q_{Eq} depending on the aggregated variable flow rate: Though the ending branches, it only circulates a small amount of the flow rate, whilst through the initial paths, the water has not yet divided in branches and has a lot more water discharge. Therefore, each path has to be calculated individually, so the design flow rates q_{Eq} are finally obtained.

Then, the initial smallest diameters out of the commercial catalogue are selected. They are assigned to the paths depending on the maximum velocity that is allowed. From this Initial Solution, diameters can only be increased in the optimization process.

At this point, the Change Gradients CG are calculated and so is the Annual Energy Cost per metre C_{E1} , and the process of finding what is more economical (building or pumping) can begin: The branch with the lowest (cheapest) CG (building cost) is proposed to be enlarged. If its CG is smaller than C_{E1} , the pipe is increased its diameter, the head losses are reduced and therefore the pumping head is diminished. Then, the branch with the lowest CG is once again identified, and the economical comparison is repeated, conforming a new iteration in the design

process. In the Case Study Chapter, Optimisation Section, there are explained certain rules in the iterative process in order to not surpass the pressure margin of any of the nodes.

The optimisation process of the case study took 19 iterations to find the optimal solution. On the 20th iteration, C_{E1} was bigger than the CG, meaning that no more diameters could be engrossed, and that, therefore, the optimum solution had been obtained. The entire case study was resolved using Microsoft Excel for the calculations.

Verification of the results

Finally, a verification of the Equivalent Flow Rate Concept was carried out: With the optimised final diameters of the case study, the annual energy cost of the facility was calculated, using the variable flow regime and the Equivalent Flow Rate, giving both, the same energy cost in a year, verifying the conceptual definition of the Equivalent Flow Rate Method and its validity.

Original contributions

To summon up the novel content explained in the previous summary, in this section the original contributions of the thesis are highlighted:

- ▶ Conceptualisation and mathematical deduction of the Equivalent Flow Rate Method.
- ▶ Conceptualisation and mathematical deduction of the Equivalent Volume Method.
- ▶ Adaptation of Gradient Based design methodologies, like the Granados' System, to a variable demand regime.
- ▶ Deep study of the values of the optimum pump efficiency.
- ▶ Determination of the existence of a strong relationship between the pump efficiency and the variable flow rate.
- ▶ Determination of the non-existent relationship between the flow rate and:
 - ▶ The pumping head.
 - ▶ The speed rotation.
- ▶ Determination of the weak relationship between the flow rate and:
 - ▶ The pumping power.
 - ▶ The specific speed.
- ▶ In depth study of the influence of the pump type over the pump efficiency.
- ▶ Quantification and calculation of the pump efficiency curve through the Flow Rate.

- ▶ Simplifications of calculations in the Granados System by braking iterative process to guess the pump efficiency.
- ▶ Proposal of a new methodology that accounts for variable flow rates, as well as an in depth energy assessment throughout the entire lifespan of the utility.
- ▶ Verification of the methodology in a complex case study.

Future outlooks

Designing water supply system is a very complex duty, and it is dependent on an abundant number of variables. For the design processes to improve, a precise definition of all variables and how they affect the design is essential. There are a number of variables that are somewhat uncertainty, and they need to be studied in depth. In many occasions, this implicates collaborating with other disciplines.

This is the case of the energy price value. The study of the evolution of electrical tariffs is necessary to achieve resilient and long lasting designs of water supply systems. The better the energy price can be predicted, the closer the operational life of the facility will be to that forecasted at the design stage, protecting the users in the fore coming years.

Another key variable is the actual water demand, which so much affects the design, as proved along this work. It is of crucial importance to have an accurate picture of the expected demand of the facility. However, determining such variable is becoming an increasingly harder task, considering the speed at which cities grow. For these reasons, future research should focus on probabilistic methods to determine the demand pattern for the fore coming years, and cooperate with social studies that could help to predict population movements.

On the other hand, because technology progresses at an incredibly rapid pace, the values of the optimum pump efficiency obtained in this work should be updated after a certain amount of years, to incorporate the advancements and update the empiric curve of pump efficiency-flow rate.

Engineers face a complex task when designing water supply systems. They bring progress to society and quality to people's life. To release part of this heavy responsibility researchers have to help the cause by bringing practicality and intuitiveness, as well as finding certainty in the uncertainty and simplicity in the complexity.

References

By alphabetic order:

1. Alperovits, E.; Shamir, U.; Design of optimal water distribution systems. *Water Resources Research*, 1977, 13 (6), 885-900.
2. Babayan, A. V.; Savic, D. A.; Walters, G. A.; Kapelan, Z. S.; Robust least-cost design of water distribution networks using redundancy and integration-based methodologies. *Journal of Water Resources Planning and Management*, 2007, 133 (1), 67-77.
3. Babayan, A.; et al. Least-cost design of water distribution networks under demand uncertainty. *Journal of Water Resources Planning and Management*, 2005, vol. 131, no 5, p. 375-382.
4. Babayan, A.; Kapelan, Z.; Savic, D.; Walters, G.; Least-cost design of water distribution networks under demand uncertainty. *Journal of Water Resources Planning and Management*, 2005, 131 (5), 375-382.
5. Capelo, B.; Pérez-Sánchez, M.; Fernandes, J. F.; Ramos, H. M.; López-Jiménez, P. A.; Branco, P. C.; Electrical behaviour of the pump working as turbine in off grid operation. *Applied energy*, 2017, 208, 302-311.
6. Cisty, M.; Bajtek, Z.; Celar, L.; A two-stage evolutionary optimization approach for an irrigation system design. *Journal of Hydroinformatics*, 2016, 19 (1), 115-122.
7. Cobacho, R.; Ramírez, R.; Torres, D.; López-Jiménez, P. A.; Implementación de un modelo de predicción de fallos orientado a la gestión y estrategias de mantenimiento en redes de distribución de agua potable. *Ingeniería del Agua*, 2019 , 23(4), 247-258.
8. Colglazier, W. Sustainable development agenda: 2030. *Science*, 2015, 349, 1048–1050.
9. Costa, A.; De Medeiros, J.; Pessoa, F.; Optimization of pipe networks including pumps by simulated annealing. *Brazilian Journal of Chemical Engineering*, 2000, 17 (4-7), 887-896.
10. Creaco, E.; Franchini, M.; Walski, T. M. Taking account of uncertainty in demand growth when phasing the construction of a water distribution network. *Journal of Water Resources Planning and Management*, 2015, vol. 141, no 2, p. 04014049.

11. Cubillo, F.; Martínez-Codina, A.; A metric approach to measure resilience in water supply systems. *Journal of Applied Water Engineering and Research*, 2019, vol. 7, no 1, p. 67-78.
12. Dandy, G.; Hewitson, C.; Optimizing hydraulics and water quality in water distribution networks using genetic algorithms. In *Building Partnerships*, 2000; pp 1-10.
13. Dziedzic, R.; Karney, B. W.; Cost gradient-based assessment and design improvement technique for water distribution networks with varying loads. *Journal of Water Resources Planning and Management*, 2015, 142 (1), 04015043.
14. Featherstone, R. E.; El-Jumaily, K.; Optimal diameter selection for pipe networks. *Journal of Hydraulic Engineering* 1983, 109 (2), 221-234.
15. Filion, Y. R.; Adams, B. J.; Karney, B. W. Stochastic design of water distribution systems with expected annual damages. *Journal of Water Resources Planning and Management*, 2007, vol. 133, no 3, p. 244-252.
16. Fu, G.; Kapelan, Z.; Reed, P.; Reducing the complexity of multiobjective water distribution system optimization through global sensitivity analysis. *Journal of Water Resources Planning and Management*, 2011, 138 (3), 196-207.
17. García de Jalón S, Silvestri S, Granados A, Iglesias A. Behavioural barriers in response to climate change in agricultural communities: an example from Kenya. *Regional Environmental Change*, 2014; 15(5):851-865.
18. Garrote L, Iglesias A, Granados A, Mediero L, Martín, F.J.; Quantitative Assessment of Climate Change Vulnerability of Irrigation Demands in Mediterranean Europe. *Water Resources Management*, 2014; 29(2):325-338.
19. Garrote, L.; de Lama, B.; Martín, F.J.; Previsiones para España según los últimos estudios de cambio climático. *El cambio climático en España y sus consecuencias en el sector agua*, 2008. Universidad Rey Juan Carlos, 3-15.
20. Gessler, J.; Walski, T. M. *Water Distribution System Optimization*; Army Engineer Waterways Experiment Station Vicksburg Ms Environmental Lab. 1985.
21. Giacomoni, M. H.; Kanta, L.; Zechman, E. M. Complex adaptive systems approach to simulate the sustainability of water resources and urbanization. *Journal of Water Resources Planning and Management*, 2013, vol. 139, no 5, p. 554-564.
22. Gomes, Heber Pimentel, Et Al. Optimal dimensioning model of water distribution systems. *Water SA*, 2009, vol. 35, no 4.
23. Gonçalves, G. M.; Gouveia, L.; Pato, M. V.; An improved decomposition-based heuristic to design a water distribution network for an irrigation system. *Annals of Operations Research* 2014, 219 (1), 141-167.

24. Goulter, I.; Systems analysis in water-distribution network design: From theory to practice. *Journal of Water Resources Planning and Management*, 1992, 118 (3), 238-248.
25. Granados García, A.; Criterios para el dimensionamiento de redes de riego robustas frente a cambios en la alternativa de cultivos, 2013, Doctoral Thesis, ETSI Caminos, Canales y Puertos (UPM).
26. Granados García, I.; Compatibilidad del principio de óptimo de Bellman con los métodos secuenciales de optimización de redes de tuberías, 2015, Doctoral Thesis.
27. Granados, García, A.; Martín, F. J.; García de Jalón, S.; Iglesias, A.; Adaptation of irrigation networks to climate change: Linking robust design and stakeholder contribution. *Spanish Journal of Agricultural Research*, 2015, 13 (4), 32.
28. Granados, Granados A.; Problemas de Obras Hidráulicas, 1995.
29. Iglesias, A.; Santillán, D.; Garrote, L. On the barriers to adaption to less water under climate change: policy choices in Mediterranean countries. *Water Resources Management*, 2018, 32(15), 4819-4832.
30. Jin, X.; Zhang, J.; Gao, J.-l.; Wu, W.-y.; Multi-objective optimization of water supply network rehabilitation with non-dominated sorting genetic algorithm-II. *Journal of Zhejiang University-Science A*, 2008, 9 (3), 391-400.
31. Kadu, M. S.; Gupta, R.; Bhawe, P. R.; Optimal design of water networks using a modified genetic algorithm with reduction in search space. *Journal of Water Resources Planning and Management*, 2008, 134 (2), 147-160.
32. Kang, D.; Lansey, K.; Revisiting optimal water-distribution system design: issues and a heuristic hierarchical approach. *Journal of Water Resources Planning and Management*, 2011, 138 (3), 208-217.
33. Kang, D.; Lansey, K.; Scenario-based robust optimization of regional water and wastewater infrastructure. *Journal of Water Resources Planning and Management*, 2012, 139 (3), 325-338.
34. Kapelan, Z. S.; Savic, D. A.; Walters, G. A.; Multiobjective design of water distribution systems under uncertainty. *Water Resources Research*, 2005, 41 (11).
35. Kim, J. H.; Mays, L. W.; Optimal rehabilitation model for water-distribution systems. *Journal of Water Resources Planning and Management*, 1994, 120 (5), 674-692.
36. Kuriqi, A.; Pinheiro, A. N.; Sordo-Ward, A.; Garrote, L.; Influence of hydrologically based environmental flow methods on flow alteration and energy production in a run-of-river hydropower plant. *Journal of Cleaner Production*, 2019, 232, 1028-1042.

37. Kuriqi, A.; Pinheiro, A. N.; Sordo-Ward, A.; Garrote, L.; Trade-off between environmental flow policy and run-of-river hydropower generation in Mediterranean climate. *Eur. Water*, 2017, 60, 123-130.
38. Lama Pedrosa, B. Metodología de evaluación e identificación de políticas de adaptación al cambio climático en la gestión de recursos hídricos, 2011 Doctoral Thesis, Universidad Politécnica de Madrid.
39. López Jiménez, A. P.; Espert Alemany, V.; Carlos Alberola, M.; Martínez Solano, F. J.; Metodología para la calibración de modelos de calidad de aguas. *Ingeniería del agua*, 2003, 10(4), 501-516.
40. Maier, H.; Kapelan, Z.; Kasprzyk, J.; Matott, L.; 2015. Thematic issue on evolutionary algorithms in water resources.
41. Mala-Jetmarova, H.; Sultanova, N.; Savic, D.; Lost in optimisation of water distribution systems? A literature review of system operation. *Environmental Modelling & Software*, 2017, 93, 209-254.
42. Marchi, A.; Dandy, G.; Wilkins, A.; Rohrlach, H.; Methodology for comparing evolutionary algorithms for optimization of water distribution systems. *Journal of Water Resources Planning and Management* 2012, 140 (1), 22-31.
43. Marques, J.; Cunha, M.; Savic, D.; Using real options in the optimal design of water distribution networks. *Journal of Water Resources Planning and Management*, 2015, vol. 141, no 2, p. 04014052.
44. Marques, J.; Cunha, M.; Savić, D.; Using real options in the optimal design of water distribution networks. *Journal of Water Resources Planning and Management*, 2014, 141 (2), 04014052.
45. Martín, F.J.; Garrote De Marcos, Luis M.; Granados García, A. Determinación de niveles de explotación de embalses para compatibilizar la gestión de avenidas y sequías. *Ingeniería civil*, 2006, vol. 144, p. 97-104.
46. Martín, F.J.; Marcos, L. G. d.; Dimensionamiento y Optimización de Obras Hidráulicas. 4th ed.; 2013.
47. Martin-Candilejo, A.; Santillán, D.; Garrote, L. Pump Efficiency Analysis for Proper Energy Assessment in Optimization of Water Supply Systems. *Water*, 2019, 12, 132.
48. Martin-Candilejo, A.; Santillán, D.; Iglesias, A.; Garrote, L. Optimization of the Design of Water Distribution Systems for Variable Pumping Flow Rates. *Water*, 2020, 12, 359.
49. Martínez Alonso, R.; Evaluación de los caudales de punta en redes colectivas de riego a la demanda, 2003, Doctoral Thesis.
50. Martínez Codina, A; Metodología de análisis estadístico de roturas en redes de distribución de agua, 2015, Doctoral Thesis.

51. Martínez-Codina, Á.; Cueto-Felgueroso, L.; Castillo, M.; Garrote, L.; Use of pressure management to reduce the probability of pipe breaks: A Bayesian approach. *Journal of Water Resources Planning and Management*, 2015, 141(9), 04015010.
52. Martínez-Codina, Á.; Saavedra, C.; Cueto-Felgueroso, L.; Garrote, L.; Influencia de la presión en roturas de tubería de redes de distribución. *Tecnología y ciencias del agua*, 2016, 7(4), 25-39.
53. Mediero L.; Santillán D, Garrote L, Granados A. Detection and attribution of trends in magnitude, frequency and timing of floods in Spain. *Journal of Hydrology*, 2014; 517:1072-1088.
54. Montalvo, Idel, et al. A diversity-enriched variant of discrete PSO applied to the design of water distribution networks. *Engineering Optimization*, 2008, vol. 40, no 7, p. 655-668.
55. Ostfeld, A.; Optimal design and operation of multiquality networks under unsteady conditions. *Journal of Water Resources Planning and Management* 2005, 131 (2), 116-124.
56. Ostfeld, A.; Tubaltzev, A.; Ant colony optimization for least-cost design and operation of pumping water distribution systems. *Journal of Water Resources Planning and Management*, 2008, 134 (2), 107-118.
57. Perelman, L.; Ostfeld, A.; An adaptive heuristic cross-entropy algorithm for optimal design of water distribution systems. *Engineering Optimization*, 2007, 39 (4), 413-428.
58. Perelman, L.; Ostfeld, A.; Salomons, E.; Cross entropy multiobjective optimization for water distribution systems design. *Water Resources Research*, 2008, 44 (9).
59. Pérez-Sánchez, M.; López-Jiménez, P. A.; Ramos, H. M.; Modified affinity laws in hydraulic machines towards the best efficiency line. *Water Resources Management*, 2018, 32 (3), 829-844.
60. Pérez-Sánchez, M.; López-Jiménez, P.; Ramos, H.; PATs Operating in Water Networks under Unsteady Flow Conditions: Control Valve Manoeuvre and Overspeed Effect, *Water*, 2018, 10(4), 529.
61. Pérez-Sánchez, M.; Sánchez-Romero, F. J.; Ramos, H. M.; López Jiménez, P. A. Calibrating a flow model in an irrigation network: Case study in Alicante, Spain. *Spanish Journal of Agricultural Research*, 2017 (Online), 15(1), 1-13.
62. Pérez-Sánchez, M.; Sánchez-Romero, F. J.; Ramos, H. M.; López-Jiménez, P. A. Energy recovery in existing water networks: Towards greater sustainability. *Water*, 2017, 9(2), 97.

63. Pérez-Sánchez, M.; Sánchez-Romero, F.-J.; López-Jiménez, P. A.; Huella energética del agua en función de los patrones de consumo en redes de distribución. *Ingeniería del agua*, 2017, 21 (3), 197-212.
64. Ramirez, R.; Torres, D.; López-Jimenez, P. A.; Cobacho, R. A Front-Line and Cost-Effective Model for the Assessment of Service Life of Network Pipes. *Water*, 2020, 12(3), 667.
65. Ramos, H. M.; McNabola, A.; López-Jiménez, P. A.; Pérez-Sánchez, M. Smart water management towards future water sustainable networks. *Water*, 2020, 12(1), 58.
66. Reca, Juan; Martínez, J; Genetic algorithms for the design of looped irrigation water distribution networks. *Water Resources Research*, 2006, vol. 42, no 5.
67. Reed, P. M.; Hadka, D.; Herman, J. D.; Kasprzyk, J. R.; Kollat, J. B.; Evolutionary multiobjective optimization in water resources: The past, present, and future. *Advances in Water Resources*, 2013, 51, 438-456.
68. Romero Marrero, L. E.; Pérez-Sánchez, M.; López Jiménez, P. A.; Water implications in Mediterranean irrigation networks: Problems and solutions. *International Journal of Energy and Environment*, 2017, 8(1), 83-96.
69. Roshani, E.; Filion, Y.; Event-based approach to optimize the timing of water main rehabilitation with asset management strategies. *Journal of Water Resources Planning and Management*, 2013, 140 (6), 04014004.
70. Samani, H. M.; Mottaghi, A.; Optimization of water distribution networks using integer linear programming. *Journal of Hydraulic Engineering*, 2006, 132 (5), 501-509.
71. Sanchis, R.; Díaz-Madroñero, M.; López-Jiménez, P. A.; Pérez-Sánchez, M.; Solution Approaches for the Management of the Water Resources in Irrigation Water Systems with Fuzzy Costs. *Water*, 2019, 11 (12), 2432.
72. Santillán, D.; Garrote, L.; Iglesias, A.; Sotes, V. Climate change risks and adaptation: new indicators for Mediterranean viticulture. *Mitigation and Adaptation Strategies for Global Change*, 2019, 1-19.
73. Santillán, D.; Iglesias, A.; La Jeunesse, I.; Garrote, L.; Sotes, V.; Vineyards in transition: a global assessment of the adaptation needs of grape producing regions under climate change. *Science of the Total Environment*, 2019, 657, 839-852.
74. Santillán, D.; Sotés, V.; Iglesias, A.; Garrote, L.; Adapting viticulture to climate change in the Mediterranean region: Evaluations accounting for spatial differences in the producers-climate interactions. In *BIO Web of Conferences*, 2019, (Vol. 12, p. 01001). EDP Sciences.

75. Schwartz, R.; Housh, M.; Ostfeld, A.; Least-cost robust design optimization of water distribution systems under multiple loading. *Journal of Water Resources Planning and Management*, 2016, 142 (9), 04016031.
76. Shokoohi, M.; Tabesh, M.; Nazif, S.; Dini, M.; Water quality based multi-objective optimal design of water distribution systems. *Water Resources Management*, 2017, 31 (1), 93-108.
77. Sordo-Ward, A.; Granados, A.; Iglesias, A.; Garrote, L.; Bejarano, M. D.; Adaptation effort and performance of water management strategies to face climate change impacts in six representative basins of Southern Europe, *Water*, 2019, 11(5), 1078.
78. Sordo-Ward, Á.; Granados, I.; Martín, F.J.; Garrote, L. Impact of hydrological uncertainty on water management decisions, *Water Resources Management*, 2016, 30 (14), 5535-5551.
79. Spiliotis, M.; Iglesias, A.; Garrote, L.; A Meta-multicriteria Approach to Estimate Drought Vulnerability Based on Fuzzy Pattern Recognition. In *International Conference on Engineering Applications of Neural Networks*, 2019 (pp. 349-360), Springer, Cham.
80. Spiliotis, M.; Papadopoulos, B. K.; A hybrid fuzzy probabilistic assessment of the extreme hydrological events. In *AIP Conference Proceedings*, 2018 (Vol. 1978, No. 1, p. 290011). AIP Publishing.
81. Spiliotis, M.; Tsakiris, G.; Incorporating uncertainty in the design of water distribution systems. *Ewra*, 2017.
82. Spiliotis, M.; Tsakiris, G.; Minimum cost irrigation network design using interactive fuzzy integer programming. *Journal of Irrigation and Drainage Engineering*, 2007, 133 (3), 242-248.
83. Stokes, C. S.; Maier, H. R.; Simpson, A. R.; Effect of storage tank size on the minimization of water distribution system cost and greenhouse gas emissions while considering time-dependent emissions factors. *Journal of Water Resources Planning and Management*, 2015, 142 (2), 04015052.
84. Stokes, C. S.; Simpson, A. R.; Maier, H. R.; A computational software tool for the minimization of costs and greenhouse gas emissions associated with water distribution systems. *Environmental Modelling & Software*, 2015, 69, 452-467.
85. Todini, E.; Looped water distribution networks design using a resilience index based heuristic approach. *Urban water*, 2000, vol. 2, no 2, p. 115-122.
86. Tolson, B.; et al. Genetic algorithms for reliability-based optimization of water distribution systems. *Journal of Water Resources Planning and Management*, 2004, vol. 130, no 1, p. 63-72.
87. Tsakiris, G.; Spiliotis, M.; Fuzzy linear programming for problems of water allocation under uncertainty. *European Water*, 2004, 7 (8), 25-37.

88. Tsakiris, G.; Spiliotis, M.; Uncertainty in the analysis of urban water supply and distribution systems. *Journal of Hydroinformatics*, 2017, 19 (6), 823-837.
89. Vamvakieridou-Lyroudia, L.; Walters, G.; Savic, D.; Fuzzy multiobjective optimization of water distribution networks. *Journal of Water Resources Planning and Management*, 2005, 131 (6), 467-476.
90. Van Dijk, M.; van Vuuren, S. J.; Van Zyl, J. E.; Optimising water distribution systems using a weighted penalty in a genetic algorithm. *Water SA*, 2008, 34 (5), 537-548.
91. Vicente, D.; Garrote, L.; Sánchez, R.; Santillán, D.; Pressure management in water distribution systems: Current status, proposals, and future trends. *Journal of Water Resources Planning and Management*, 2015, 142 (2), 04015061.
92. Vicente, D.; Garrote, L.; Sánchez, R.; Santillán, D.; Pressure management in water distribution systems: Current status, proposals, and future trends. *Journal of Water Resources Planning and Management*, 2015, 142 (2), 04015061.
93. Walski, T. M.; Brill Jr, E. D.; Gessler, J.; Goulter, I. C.; Jeppson, R. M.; Lansey, K.; Lee, H.-L.; Liebman, J. C.; Mays, L.; Morgan, D. R.; Battle of the network models: Epilogue. *Journal of Water Resources Planning and Management*, 1987, 113 (2), 191-203.
94. Walski, Thomas M. The wrong paradigm—Why water distribution optimization doesn't work. *Journal of Water Resources Planning and Management*, 2001, vol. 127, no 4, p. 203-205.
95. Wu, W.; Maier, H. R.; Simpson, A. R.; Single-objective versus multiobjective optimization of water distribution systems accounting for greenhouse gas emissions by carbon pricing. *Journal of Water Resources Planning and Management*, 2009, 136 (5), 555-565.
96. Wu, W.; Simpson, A. R.; Maier, H. R.; Accounting for greenhouse gas emissions in multiobjective genetic algorithm optimization of water distribution systems. *Journal of Water Resources Planning and Management*, 2009, 136 (2), 146-155.
97. Wu, W.; Simpson, A. R.; Maier, H. R.; Marchi, A.; Incorporation of variable-speed pumping in multiobjective genetic algorithm optimization of the design of water transmission systems. *Journal of Water Resources Planning and Management*, 2011, 138 (5), 543-552.
98. Wu, W.; Simpson, A. R.; Maier, H. R.; Multi-objective genetic algorithm optimisation of water distribution systems accounting for sustainability. *Proceedings of Water Down*, 2008, 1750.
99. Wu, W.; Simpson, A. R.; Maier, H. R.; Sensitivity of optimal tradeoffs between cost and greenhouse gas emissions for water distribution systems to electricity tariff and generation. *Journal of Water Resources Planning and Management*, 2011, 138 (2), 182-186.

100. Zecchin, Aaron C.; et al. Ant colony optimization applied to water distribution system design: Comparative study of five algorithms. *Journal of Water Resources Planning and Management*, 2007, vol. 133, no 1, p. 87-92.
101. Zheng, F.; Simpson, A.; Zecchin, A.; Improving the efficiency of multi-objective evolutionary algorithms through decomposition: An application to water distribution network design. *Environmental Modelling & Software*, 2015, 69, 240-252.

Annex: Case Study

Description of the case study

Topography

The network's consists of 12 paths, 13 nots and 5 different agricultural Sectors (S_A , S_B , S_C , S_D and S_E). The topography of the net is specified in the following **table A.1.**

Path	Initial not height m	Ending not height m	Length of the path m
1	610	671	1500
2	671	676	2000
3	676	650	500
4	650	670	4500
5	670	675	3000
6	670	655	4500
7	650	630	3000
8	630	622	3000
9	622	590	3000
10	590	595	3500
11	630	640	4000
12	640	625	5500

Table A.1.

All calculations of the case study were made using Microsoft Excel.

Water Demand

The characteristics of the water demand of the network can be summarised in the following **table A.2**.

Calculation of the Equivalent Flow Rate									
Plantation Sector									
	A	B	C	D	E	A+B	C+D	C+D+E	A+B+C +D+E
Path									
	5	6	9, 10	None	11, 12	4	8	7	1, 2, 3
Month	Flow rate, q_k								
k	m ³ /s	m ³ /s	m ³ /s	m ³ /s	m ³ /s	m ³ /s	m ³ /s	m ³ /s	m ³ /s
January	0.05	0.04	0.07	0.01	0.04	0.09	0.08	0.12	0.21
February	0.10	0.04	0.09	0.02	0.08	0.14	0.11	0.19	0.33
March	0.07	0.03	0.11	0.03	0.09	0.09	0.14	0.23	0.33
April	0.10	0.04	0.12	0.05	0.09	0.14	0.16	0.26	0.40
May	0.10	0.06	0.18	0.07	0.10	0.16	0.25	0.35	0.51
June	0.18	0.05	0.20	0.11	0.11	0.22	0.31	0.42	0.65
July	0.18	0.07	0.20	0.09	0.12	0.26	0.29	0.41	0.66
August	0.17	0.06	0.17	0.08	0.11	0.22	0.25	0.36	0.58
September	0.11	0.03	0.14	0.07	0.09	0.14	0.22	0.31	0.46
October	0.05	0.02	0.07	0.07	0.07	0.07	0.14	0.21	0.28
November	0.05	0.02	0.06	0.05	0.06	0.07	0.10	0.17	0.23
December	0.05	0.02	0.01	0.02	0.05	0.06	0.03	0.08	0.15
Average	0.10	0.04	0.12	0.06	0.09	0.14	0.17	0.26	0.40
Maximum	0.18	0.07	0.20	0.11	0.12	0.26	0.31	0.42	0.66
Design Value: Equivalent Flow Rate									
Equivalent	0.13	0.05	0.16	0.08	0.09	0.18	0.23	0.32	0.49

Table A.2.

Initial Solution

Maximum velocity

The following table shows the available pipe diameter of a commercial manufacturer, along with the price of the pipe per 1000 metres, in the second column. The maximum velocity is defined as $v_{\max} [m/s] = 2 + \emptyset$, and it is shown in the 3rd column. It will serve to determinate the minimum diameters of the net and work as the initial solution.

Commercial Pipe Diameters and Maximum Velocity			
φ	Price	$V_{\max}$	$Q_{\max}$
mm	€/1000 m	m/s	m ³ /s
1300	1,760,872	3.3	4.4
1200	1,369,891	3.2	3.6
1100	1,048,281	3.1	2.9
1000	786,380	3.0	2.4
900	630,871	2.9	1.8
800	545,380	2.8	1.4
700	416,600	2.7	1.0
600	347,435	2.6	0.7
500	281,487	2.5	0.5
450	249,719	2.5	0.4
400	218,755	2.4	0.3
350	169,241	2.4	0.2
300	127,178	2.3	0.2
250	96,133	2.3	0.1
200	76,001	2.2	0.1

Table A.3.

Initial diameters

The commercial pipe diameters that strictly meet the maximum water velocity are shown in the following **table A.4**. From these values, the optimisation is initiated and the pipe's diameters are enlarged.

Network characteristics					
Path	Previous Path	Extra costs Sulphates and water hammer	Length m	Flow Rates m ³ /s	Ending Heights m
1	0	1.20	1500	0.49	671
2	1	1.00	2000	0.49	676
3	2	1.00	500	0.49	650
4	3	1.00	4500	0.18	670
5	4	1.00	3000	0.13	675
6	4	1.00	4500	0.05	655
7	3	1.00	3000	0.32	630
8	7	1.00	3000	0.23	622
9	8	1.00	3000	0.16	590

10	9	1.00	3500	0.16	595
11	7	1.20	4000	0.09	640
12	11	1.0	5500	0.09	625

Table A.4.

Energy Cost calculation

The following table shows the results of the total cost per meter of pumping head during the entire lifespan of the utility. It has been calculated using the results of the pump efficiency study, in order to obtain the value of the expected pump efficiency.

Total Cost per meter of pumping head		
Total C1	6,1265.3	€/m
Annual c1	5,161.9	€/m*year
Volume	12,542,400	m3/year
Energy price: p_e	0.12	€/kWh
$\mu_B \cdot \mu_M$	0.79	%
Pumping efficiency		
Average μ_B	0.85	%
Maximum μ_B	0.88	%
μ_M	0.93	%
Discount factor		
f_a	11.87	
i: Interest rate	0.05	
Life span of the utility	20	years
Construction years	1	years

Table A.5.

Iterations

The iterations have been programmed in Microsoft Excel. Each iteration follows the next steps and nomenclature:

Programming Procedure	
1	See where Pressure Margin PM =0
2	Introduce those paths where PM=0 in the Identification of the Change Alternatives box
3	If Several Critical Paths CP : See the Cheapest Alternative combination
3. 1	Generate the Change Alternatives CA
3. 2	Identify the cheapest CA
4	Check if it is cheaper to increase the size of the pipe, or pump the head loss.
The paths are marked as:	
5	1 Affected Path
	DS Path Down Stream
	(nothing) Path to Change

Table A.6.

Then, the iterations are carried out. It took 19 iterations to find the optimum diameter. On the 20th iteration, the energy cost was cheaper than the Change Gradient of the cheapest alternative, and therefore, the optimisation was finished. **Table A.7..** shows the compilation of Change Alternatives CA that was proposed for each iteration. As it can be seen, most of the CA involved changing the pipe diameter of several paths at the same time, and the Change Gradients had to be summed.

Generation of Change Alternatives

Iteration Number:

2 3 and 4 5 until 19

Change
Alternative CA: Paths to Change in each CA:

CA 1	1	1	1
CA 2	2	2	2
CA 3	3	3	3
CA 4	6	6	5
	7	7	6
CA 5	6	4	4
	11	7	7
CA 6	6	6	5
	12	10	6
		12	10
CA 7			12
	4	6	5
	7	10	6
		11	10
CA 8			11
	4	6	5
	11	9	6
		12	9
CA 9			12
	4	6	5
	12	9	6
		11	9
CA 10			11
		6	5
		8	6
		12	8
CA 11			12
		6	5
		8	6
		11	8
CA 12			11
		4	4
		10	8
CA 13		12	11
		4	4
		10	8
CA 14		11	12
		4	4
		9	9
		12	11

CA15	4	4
	9	9
	11	12
CA16	4	4
	8	10
	12	11
CA17	4	4
	8	10
	11	12

Table A.7.

The next **table A.8.** shows the resulting Change Alternatives selected for each iteration, along with the Change Gradients. These were the cheapest alternative to change, and therefore, the selected paths to be enlarged. Most iterations had several paths changed at the same time, as it can be seen.

Identification of the paths to change					
Iteration	Paths				CG
1	12				2,307
2	6	12			3,793
3	6	9	12		7,217
4	6	9	11		7,678
5	5	6	9	11	12,299
6	5	6	10	11	12,299
7	4	10	11		13,683
8	4	7			14,411
9	4	7			17,349
10	2				22,301
11	3				22,301
12	4	10	12		23,022
13	1				26,761
14	5	6	10	12	27,696
15	5	6	8	12	30,784
16	5	6	8	12	36,124
17	4	8	12		37,438
18	4	7			39,178
19	5	6	7		44,805
20	5	6	7		65,399

Table A.8.

The following **tables A.9.1 - A.9.21**. show the calculations of the iterations required for the design process.

Iteration 1

Iteration number									
1	Variation of the pumping head calculation					Pumping head [m]: 823.18			Remaining $\Delta\Delta h$
Paths to change	φ Mixed mm	φ after change mm	$\Delta \Delta h$ m	Affected paths	PM m	φ Mixed mm	φ mm	PM m	m
		600	-	1	205.4		600	143.9	
		600	-	1	189.4		600	127.8	
		600	-	1	212.6		600	151.0	
		350	-	1	134.2		350	72.7	
		300	-	1	80.5		300	18.9	
		200	-	1	61.6		200	0.0	
		450	-	1	200.6		450	139.0	
		400	-	1	178.0		400	116.4	
		300	-	1	144.2		300	82.7	
		300	-	1	62.5		300	1.0	
		250	-	1	104.0		250	42.5	
Change		300	74.01		-	300	250	0.0	12.4
Variation of the Pumping Head					61.57				
The path changes					Partially				

Iteration: 1		Identification of the Critical Paths						
Increase the size of the pipe, or pump the head loss?								Increase
How many Critical Paths?		One CP						
PM= 0	12	12	11	7	3	2	1	12
in path:	GC	2,307	2,769	6,920	22,301	22,301	26,761	2,307

Table A.9.1

Iteration 2

2 Iteration number									
Paths to change	Variation of the pumping head calculation					Pumping head [m]: 822.21			Remaining $\Delta\Delta h$ m
	φ Mixed	φ after change	$\Delta \Delta h$	Affected paths	PM	φ Mixed	φ	PM	
	mm	mm	m		m		mm	m	
Change		600	-	1	143.9		600	142.9	
		600	-	1	127.8		600	126.8	
		600	-	1	151.0		600	150.1	
		350	-	1	72.7		350	71.7	
		300	-	1	18.9		300	18.0	
		250	60.99		-	250	200	0.0	60.0
		450	-	1	139.0		450	138.0	
		400	-	1	116.4		400	115.5	
		300	-	1	82.7		300	81.7	
		300	-	1	1.0		300	0.0	
Change		250	-	1	42.5		250	41.5	
	300	300	12.44		-	300	250	0.0	11.5
Variation of the Pumping Head					0.97				
The path changes					Partially				

Iteration: 2		Identification of the Critical Paths					
Increase the size of the pipe, or pump the head loss?:						Increase	
How many Critical Paths?						Several CP	
PM= 0	6	6	4	3	2	1	
in path:	GC	1,485	7,490	22,301	22,301	26,761	
PM= 0	12	12	11	7	3	2	1
in path:	GC	2,307	2,769	6,920	22,301	22,301	26,761

Table A.9.2

Iteration 3

3			Iteration number							
Paths to change	Variation of the pumping head calculation					Pumping head [m]:			Remaining $\Delta\Delta h$	
	φ Mixed	φ after change	$\Delta \Delta h$	Affected paths	PM	φ Mixed	φ	PM		
	mm	mm	m		m		mm	m		
		600	-	1	142.9		600	131.4		
		600	-	1	126.8		600	115.4		
		600	-	1	150.1		600	138.6		
		350	-	1	71.7		350	60.2		
		300	-	1	18.0		300	6.5		
Change	250	250	60.02		-	250	200	0.0	48.5	
		450	-	1	138.0		450	126.6		
		400	-	1	115.5		400	104.0		
		350	36.85		-	350	300	81.7		25.4
		300	-	DS	-		300	0.0		
		250	-	1	41.5		250	30.0		
Change	300	300	11.48		-	Completo	300	0.0		
Variation of the Pumping Head					11.48					
The path changes					Partially					

Iteration: 3		Identification of the Critical Paths						
Increase the size of the pipe, or pump the head loss?:							Increase	
How many Critical Paths?							Several CP	
PM= 0	6	6	4	3	2	1		
in path:	GC	1,485	7,490	22,301	22,301	26,761		
PM= 0	10	10	9	8	7	3	2	1
in path:	GC	3,424	3,424	6,511	6,920	22,301	22,301	26,761
PM= 0	12	12	11	7	3	2	1	
in path:	GC	2,307	2,769	6,920	22,301	22,301	26,761	

Table A.9.3

Iteration 4

4 Iteration number									
Paths to change	Variation of the pumping head calculation					Pumping head [m]: 804.25			Remaining $\Delta\Delta h$ m
	φ Mixed	φ after change	$\Delta \Delta h$	Affected paths	PM	φ Mixed	φ	PM	
	mm	mm	m		m		mm	m	
Change	250	600	-	1	131.4		600	124.9	42.1
		600	-	1	115.4		600	108.9	
		600	-	1	138.6		600	132.1	
		350	-	1	60.2		350	53.7	
		300	-	1	6.5		300	0.0	
Change	350	250	48.55		-	250	200	0.0	18.9
		450	-	1	126.6		450	120.1	
		400	-	1	104.0		400	97.5	
Change	300	350	25.38		-	350	300	81.7	47.3
		300	-	DS	-		300	0.0	
		300	53.82		-	300	250	30.0	
		300	-	DS	-		300	0.0	
Variation of the Pumping Head					6.49				
The path changes					Partially				

Iteration: 4		Identification of the Critical Paths						
Increase the size of the pipe, or pump the head loss?:							Increase	
How many Critical Paths?							Several CP	
PM= 0	6	6	4	3	2	1		
in path:	GC	1,485	7,490	22,301	22,301	26,761		
PM= 0	10	10	9	8	7	3	2	1
in path:	GC	3,424	3,424	6,511	6,920	22,301	22,301	26,761
PM= 0	12	12	11	7	3	2	1	
in path:	GC	9,170	2,769	6,920	22,301	22,301	26,761	

Table A.9.4

Iteration 5

5 Iteration number									
Paths to change	Variation of the pumping head calculation					Pumping head [m]: 785.36			Remaining $\Delta\Delta h$
	φ Mixed	φ after change	$\Delta \Delta h$	Affected paths	PM	φ Mixed	φ	PM	
	mm	mm	m		m		mm	m	m
		600	-	1	124.9		600	106.1	
		600	-	1	108.9		600	90.0	
		600	-	1	132.1		600	113.2	
		350	-	1	53.7		350	34.8	
Change		350	27.31		-	350	300	0.0	8.4
Change	250	250	42.06		-	250	200	0.0	23.2
		450	-	1	120.1		450	101.2	
		400	-	1	97.5		400	78.6	
Change	350	350	18.89		-	Completed	350	81.7	
		300	-	DS	-		300	0.0	
Change	300	300	47.34		-	300	250	30.0	28.4
		300	-	DS	-		300	0.0	

Iteration: 5		Identification of the Critical Paths						
Increase the size of the pipe, or pump the head loss?:							Increase	
How many Critical Paths?							Several CP	
PM= 0	5	5	4	3	2	1		
in path:	GC	4,621	7,490	22,301	22,301	26,761		
PM= 0	6	6	4	3	2	1		
in path:	GC	1,485	7,490	22,301	22,301	26,761		
PM= 0	10	10	9	8	7	3	2	1
in path:	GC	3,424	3,424	6,511	6,920	22,301	22,301	26,761
PM= 0	12	12	11	7	3	2	1	
in path:	GC	9,170	2,769	6,920	22,301	22,301	26,761	

Table A.9.5

Iteration 6

Iteration number									
6									
Paths to change	Variation of the pumping head calculation					Pumping head [m]: 776.94			Remaining $\Delta\Delta h$
	φ Mixed	φ after change	$\Delta \Delta h$	Affected paths	PM	φ Mixed	φ	PM	
	mm	mm	m		m		mm	m	
Change	350	600	-	1	106.1	Completed	600	97.6	14.8
		600	-	1	90.0		600	81.6	
		600	-	1	113.2		600	104.8	
		350	-	1	34.8		350	26.4	
		350	8.42		-		350	0.0	
Change	250	250	23.17		-	250	200	0.0	
Change	300	450	-	1	101.2	Completed	450	92.8	34.6
		400	-	1	78.6		400	70.2	
		350	-	1	81.7		350	73.3	
		350	43.00		-		350	300	
		300	28.45		-		300	250	
Change	300	300	-	DS	-	300	300	0.0	20.0
Variation of the Pumping Head					8.42				
The path changes					Partially				

Iteration: 6		Identification of the Critical Paths						
Increase the size of the pipe, or pump the head loss?:							Increase	
How many Critical Paths?							Several CP	
PM= 0	5	5	4	3	2	1		
in path:	GC	4,621	7,490	22,301	22,301	26,761		
PM= 0	6	6	4	3	2	1		
in path:	GC	1,485	7,490	22,301	22,301	26,761		
PM= 0	10	10	9	8	7	3	2	1
in path:	GC	3,424	10,091	6,511	6,920	22,301	22,301	26,761
PM= 0	12	12	11	7	3	2	1	
in path:	GC	9,170	2,769	6,920	22,301	22,301	26,761	

Table A.9.6

Iteration 7

7 Iteration number									
Paths to change	Variation of the pumping head calculation					Pumping head [m]: 756.91			Remaining $\Delta\Delta h$
	φ Mixed	φ after change	$\Delta \Delta h$	Affected paths	PM	φ Mixed	φ	PM	
	mm	mm	m		m		mm	m	m
Change		600	-	1	97.6		600	77.6	
		600	-	1	81.6		600	61.5	
		600	-	1	104.8		600	84.8	
		400	29.75	-	-	400	350	26.4	9.7
		350	-	DS	-		350	0.0	
	250	200	-	DS	-	250	200	0.0	14.8
		450	-	1	92.8		450	72.7	
Change		400	-	1	70.2		400	50.2	
		350	-	1	73.3		350	53.3	
	350	350	34.58	-	-	350	300	0.0	14.5
Change	300	300	20.03	-	-	Completed	300	30.0	
		300	-	DS	-		300	0.0	
Variation of the Pumping Head					20.03				
The path changes					Partially				

Iteration: 7		Identification of the Critical Paths						
Increase the size of the pipe, or pump the head loss?:							Increase	
How many Critical Paths?							Several CP	
PM= 0	5	5	4	3	2	1		
in path:	GC	13,617	7,490	22,301	22,301	26,761		
PM= 0	6	6	4	3	2	1		
in path:	GC	1,485	7,490	22,301	22,301	26,761		
PM= 0	10	10	9	8	7	3	2	1
in path:	GC	3,424	10,091	6,511	6,920	22,301	22,301	26,761
PM= 0	12	12	11	7	3	2	1	
in path:	GC	9,170	2,769	6,920	22,301	22,301	26,761	

Table A.9.7

Iteration 8

8						Iteration number				
Paths to change	Variation of the pumping head calculation					Pumping head [m]: 747.19			Remaining $\Delta\Delta h$	
	φ Mixed	φ after change	$\Delta \Delta h$	Affected paths	PM	φ Mixed	φ	PM		
	mm	mm	m		m		mm	m		
Change	400	600	-	1	77.6	Completed	600	67.9	14.8	
		600	-	1	61.5		600	51.8		
		600	-	1	84.8		600	75.1		
		400	9.72	-	-		400	26.4		
Change	250	350	-	DS	-	250	350	0.0	4.1	
		200	-	DS	-		200	0.0		
		500	13.77	-	-		500	450		72.7
		400	-	DS	-		400	50.2		
		350	-	DS	-		350	53.3		
		350	-	DS	-		350	300		0.0
		300	-	DS	-		300	30.0		
		300	-	DS	-		300	0.0		
Variation of the Pumping Head					9.72					
The path changes					Partially					

Iteration: 8		Identification of the Critical Paths						
Increase the size of the pipe, or pump the head loss?:							Increase	
How many Critical Paths?							Several CP	
PM= 0	5	5	4	3	2	1		
in path:	GC	13,617	7,490	22,301	22,301	26,761		
PM= 0	6	6	4	3	2	1		
in path:	GC	1,485	7,490	22,301	22,301	26,761		
PM= 0	10	10	9	8	7	3	2	1
in path:	GC	3,424	10,091	6,511	6,920	22,301	22,301	26,761
PM= 0	12	12	11	7	3	2	1	
in path:	GC	9,170	11,004	6,920	22,301	22,301	26,761	

Table A.9.8

Iteration 9

9 Iteration number									
Paths to change	Variation of the pumping head calculation					Pumping head [m]: 743.14			Remaining $\Delta\Delta h$
	φ Mixed mm	φ after change mm	$\Delta \Delta h$ m	Affected paths	PM m	φ Mixed	φ mm	PM m	
Change		600	-	1	67.9		600	63.8	
		600	-	1	51.8		600	47.8	
		600	-	1	75.1		600	71.0	
		450	13.36	-	-	450	400	26.4	9.3
		350	-	DS	-		350	0.0	
Change	250	200	-	DS	-	250	200	0.0	14.8
	500	500	4.05	-	-	Completed	500	72.7	
		400	-	DS	-		400	50.2	
		350	-	DS	-		350	53.3	
	350	300	-	DS	-	350	300	0.0	14.5
		300	-	DS	-		300	30.0	
		300	-	DS	-		300	0.0	
Variation of the Pumping Head					4.05				
The path changes					Partially				

Iteration: 9		Identification of the Critical Paths						
Increase the size of the pipe, or pump the head loss?:							Increase	
How many Critical Paths?							Several CP	
PM= 0	5	5	4	3	2	1		
in path:	GC	13,617	10,428	22,301	22,301	26,761		
PM= 0	6	6	4	3	2	1		
in path:	GC	1,485	10,428	22,301	22,301	26,761		
PM= 0	10	10	9	8	7	3	2	1
in path:	GC	3,424	10,091	6,511	6,920	22,301	22,301	26,761
PM= 0	12	12	11	7	3	2	1	
in path:	GC	9,170	11,004	6,920	22,301	22,301	26,761	

Table A.9.10

Iteration 10

Iteration number									
10									
Paths to change	Variation of the pumping head calculation					Pumping head [m]: 736.94			Remaining $\Delta\Delta h$
	φ Mixed	φ after change	$\Delta \Delta h$	Affected paths	PM	φ Mixed	φ	PM	
	mm	mm	m		m		mm	m	
Change		600	-	1	63.8		600	57.6	
		700	6.20		-		700	47.8	
		600	-	DS	-		600	71.0	
	450	400	-	DS	-	450	400	26.4	9.3
		350	-	DS	-		350	0.0	
	250	200	-	DS	-	250	200	0.0	14.8
		500	-	DS	-		500	72.7	
		400	-	DS	-		400	50.2	
		350	-	DS	-		350	53.3	
	350	300	-	DS	-	350	300	0.0	14.5
		300	-	DS	-		300	30.0	
		300	-	DS	-		300	0.0	
Variation of the Pumping Head					6.20				
The path changes					Fully				

Iteration: 10		Identification of the Critical Paths						
Increase the size of the pipe, or pump the head loss?:							Increase	
How many Critical Paths?							Several CP	
PM= 0	5	5	4	3	2	1		
in path:	GC	13,617	10,428	22,301	22,301	26,761		
PM= 0	6	6	4	3	2	1		
in path:	GC	1,485	10,428	22,301	22,301	26,761		
PM= 0	10	10	9	8	7	3	2	1
in path:	GC	3,424	10,091	6,511	17,421	22,301	22,301	26,761
PM= 0	12	12	11	7	3	2	1	
in path:	GC	9,170	11,004	17,421	22,301	22,301	26,761	

Table A.9.11

Iteration 11

Iteration number									
11									
Paths to change	Variation of the pumping head calculation					Pumping head [m]: 735.39			Remaining $\Delta\Delta h$
	φ Mixed	φ after change	$\Delta \Delta h$	Affected paths	PM	φ Mixed	φ	PM	
	mm	mm	m		m		mm	m	
Change		600	-	1	57.6		600	56.1	
		700	-	1	47.8		700	46.2	
		700	1.55		-		700	71.0	
	450	400	-	DS	-	450	400	26.4	9.3
		350	-	DS	-		350	0.0	
	250	200	-	DS	-	250	200	0.0	14.8
		500	-	DS	-		500	72.7	
		400	-	DS	-		400	50.2	
		350	-	DS	-		350	53.3	
	350	300	-	DS	-	350	300	0.0	14.5
		300	-	DS	-		300	30.0	
		300	-	DS	-		300	0.0	
Variation of the Pumping Head					1.55				
The path changes					Fully				

Iteration: 11		Identification of the Critical Paths						
Increase the size of the pipe, or pump the head loss?:							Increase	
How many Critical Paths?							Several CP	
PM= 0	5	5	4	3	2	1		
in path:	GC	13,617	10,428	22,301	103,953	26,761		
PM= 0	6	6	4	3	2	1		
in path:	GC	1,485	10,428	22,301	103,953	26,761		
PM= 0	10	10	9	8	7	3	2	1
in path:	GC	3,424	10,091	6,511	17,421	22,301	103,953	26,761
PM= 0	12	12	11	7	3	2	1	
in path:	GC	9,170	11,004	17,421	22,301	103,953	26,761	

Table A.9.12

Iteration 12

Iteration number									
12									
Paths to change	Variation of the pumping head calculation					Pumping head [m]: 726.08			Remaining $\Delta\Delta h$
	φ Mixed	φ after change	$\Delta \Delta h$	Affected paths	PM	φ Mixed	φ	PM	
	mm	mm	m		m		mm	m	
Change	450	600	-	1	56.1	Completed	600	46.8	14.8
		700	-	1	46.2		700	36.9	
		700	-	1	71.0		700	61.7	
		450	9.31	-	-		450	26.4	
		350	-	DS	-		350	0.0	
		250	-	DS	-		250	0.0	
		500	-	1	72.7		500	63.4	
Change	350	400	-	1	50.2	350	400	40.8	5.2
		350	-	1	53.3		350	44.0	
		350	14.55	-	-		350	300	
		300	-	1	30.0		300	20.7	
Change		350	25.23	-	-	350	300	0.0	15.9
Variation of the Pumping Head					9.31				
The path changes					Partially				

Iteration: 12		Identification of the Critical Paths						
Increase the size of the pipe, or pump the head loss?:							Increase	
How many Critical Paths?							Several CP	
PM= 0	5	5	4	3	2	1		
in path:	GC	13,617	10,428	103,953	103,953	26,761		
PM= 0	6	6	4	3	2	1		
in path:	GC	1,485	10,428	103,953	103,953	26,761		
PM= 0	10	10	9	8	7	3	2	1
in path:	GC	3,424	10,091	6,511	17,421	103,953	103,953	26,761
PM= 0	12	12	11	7	3	2	1	
in path:	GC	9,170	11,004	17,421	103,953	103,953	26,761	

Table A.9.13

Iteration 13

Iteration number									
13									
Paths to change	Variation of the pumping head calculation					Pumping head [m]: 721.43			Remaining $\Delta\Delta h$
	φ Mixed	φ after change	$\Delta \Delta h$	Affected paths	PM	φ Mixed	φ	PM	
	mm	mm	m		m		mm	m	
Change		700	4.65	1	46.8		700	46.8	
		700	-	DS	-		700	36.9	
		700	-	DS	-		700	61.7	
		450	-	DS	-		450	26.4	
		350	-	DS	-		350	0.0	
	250	200	-	DS	-	250	200	0.0	14.8
		500	-	DS	-		500	63.4	
		400	-	DS	-		400	40.8	
		350	-	DS	-		350	44.0	
	350	300	-	DS	-	350	300	0.0	5.2
		300	-	DS	-		300	20.7	
	350	300	-	DS	-	350	300	0.0	15.9
Variation of the Pumping Head					4.65				
The path changes					Fully				

Iteration: 13		Identification of the Critical Paths						
Increase the size of the pipe, or pump the head loss?:							Increase	
How many Critical Paths?							Several CP	
PM= 0	5	5	4	3	2	1		
in path:	GC	13,617	21,757	103,953	103,953	26,761		
PM= 0	6	6	4	3	2	1		
in path:	GC	1,485	21,757	103,953	103,953	26,761		
PM= 0	10	10	9	8	7	3	2	1
in path:	GC	3,424	10,091	6,511	17,421	103,953	103,953	26,761
PM= 0	12	12	11	7	3	2	1	
in path:	GC	9,170	11,004	17,421	103,953	103,953	26,761	

Table A.9.14

Iteration 14

Iteration number									
14									
Paths to change	Variation of the pumping head calculation					Pumping head [m]: 716.19			Remaining $\Delta\Delta h$
	φ Mixed	φ after change	$\Delta \Delta h$	Affected paths	PM	φ Mixed	φ	PM	
	mm	mm	m		m		mm	m	
		700	-	1	46.8		700	41.5	
		700	-	1	36.9		700	31.7	
		700	-	1	61.7		700	56.5	
		450	-	1	26.4		450	21.2	
Change		400	10.91		-	400	350	0.0	5.7
Change	250	250	14.75		-	250	200	0.0	9.5
		500	-	1	63.4		500	58.2	
		400	-	1	40.8		400	35.6	
		350	-	1	44.0		350	38.7	
Change	350	350	5.24		-	Completed	350	0.0	
		300	-	1	20.7		300	15.5	
Change	350	350	15.92		-	350	300	0.0	10.7
Variation of the Pumping Head					5.24				
The path changes					Partially				

Iteration: 14		Identification of the Critical Paths						
Increase the size of the pipe, or pump the head loss?:							Increase	
How many Critical Paths?							Several CP	
PM= 0	5	5	4	3	2	1		
in path:	GC	13,617	21,757	103,953	103,953	124,743		
PM= 0	6	6	4	3	2	1		
in path:	GC	1,485	21,757	103,953	103,953	124,743		
PM= 0	10	10	9	8	7	3	2	1
in path:	GC	3,424	10,091	6,511	17,421	103,953	103,953	124,743
PM= 0	12	12	11	7	3	2	1	
in path:	GC	9,170	11,004	17,421	103,953	103,953	124,743	

Table A.9.15

Iteration 15

Iteration number									
15									
Paths to change	Variation of the pumping head calculation					Pumping head [m]: 710.52			Remaining $\Delta\Delta h$
	φ Mixed	φ after change	$\Delta \Delta h$	Affected paths	PM	φ Mixed	φ	PM	
	mm	mm	m		m		mm	m	m
		700	-	1	41.5		700	35.9	
		700	-	1	31.7		700	26.0	
		700	-	1	56.5		700	50.8	
		450	-	1	21.2		450	15.5	
Change	400	400	5.67		-	Completed	400	0.0	
Change	250	250	9.51		-	250	200	0.0	3.8
		500	-	1	58.2		500	52.5	
Change		450	14.27		-	450	400	35.6	8.6
		350	-	DS	-		350	38.7	
		350	-	DS	-		350	0.0	
		300	-	1	15.5		300	9.8	
Change	350	350	10.68		-	350	300	0.0	5.0
Variation of the Pumping Head					5.67				
The path changes					Partially				

Iteration: 15		Identification of the Critical Paths						
Increase the size of the pipe, or pump the head loss?:							Increase	
How many Critical Paths?							Several CP	
PM= 0	5	5	4	3	2	1		
in path:	GC	13,617	21,757	103,953	103,953	124,743		
PM= 0	6	6	4	3	2	1		
in path:	GC	1,485	21,757	103,953	103,953	124,743		
PM= 0	10	10	9	8	7	3	2	1
in path:	GC	10,091	10,091	6,511	17,421	103,953	103,953	124,743
PM= 0	12	12	11	7	3	2	1	
in path:	GC	9,170	11,004	17,421	103,953	103,953	124,743	

Table A.9.16

Iteration 16

Iteration number									
16									
Paths to change	Variation of the pumping head calculation					Pumping head [m]: 706.68			Remaining $\Delta\Delta h$
	φ Mixed	φ after change	$\Delta \Delta h$	Affected paths	PM	φ Mixed	φ	PM	
	mm	mm	m		m		mm	m	m
		700	-	1	35.9		700	32.0	
		700	-	1	26.0		700	22.2	
		700	-	1	50.8		700	46.9	
		450	-	1	15.5		450	11.7	
Change		450	4.90		-	450	400	0.0	1.1
Change	250	250	3.84		-	Completed	250	0.0	
		500	-	1	52.5		500	48.7	
Change	450	450	8.60		-	450	400	35.6	4.8
		350	-	DS	-		350	38.7	
		350	-	DS	-		350	0.0	
		300	-	1	9.8		300	5.9	
Change	350	350	5.01		-	350	300	0.0	1.2
Variation of the Pumping Head					3.84				
The path changes					Partially				

Iteration: 16		Identification of the Critical Paths						
Increase the size of the pipe, or pump the head loss?:							Increase	
How many Critical Paths?							Several CP	
PM= 0	5	5	4	3	2	1		
in path:	GC	18,958	21,757	103,953	103,953	124,743		
PM= 0	6	6	4	3	2	1		
in path:	GC	1,485	21,757	103,953	103,953	124,743		
PM= 0	10	10	9	8	7	3	2	1
in path:	GC	10,091	10,091	6,511	17,421	103,953	103,953	124,743
PM= 0	12	12	11	7	3	2	1	
in path:	GC	9,170	11,004	17,421	103,953	103,953	124,743	

Table A.9.17

Iteration 17

Iteration number									
17									
Paths to change	Variation of the pumping head calculation					Pumping head [m]: 705.51			Remaining $\Delta\Delta h$
	φ Mixed	φ after change	$\Delta \Delta h$	Affected paths	PM	φ Mixed	φ	PM	
	mm	mm	m		m		mm	m	m
Change		700	-	1	32.0		700	30.9	
		700	-	1	22.2		700	21.0	
		700	-	1	46.9		700	45.8	
		500	6.57	-	-	500	450	11.7	5.4
	450	400	-	DS	-	450	400	0.0	1.1
Change		250	-	DS	-		250	0.0	
		500	-	1	48.7		500	47.5	
	450	450	4.76	-	-	450	400	35.6	3.6
		350	-	DS	-		350	38.7	
		350	-	DS	-		350	0.0	
Change		300	-	1	5.9		300	4.8	
	350	350	1.17	-	-	Completed	350	0.0	
Variation of the Pumping Head					1.17				
The path changes					Partially				

Iteration: 17		Identification of the Critical Paths						
Increase the size of the pipe, or pump the head loss?:							Increase	
How many Critical Paths?							Several CP	
PM= 0	5	5	4	3	2	1		
in path:	GC	18,958	21,757	103,953	103,953	124,743		
PM= 0	6	6	4	3	2	1		
in path:	GC	8,426	21,757	103,953	103,953	124,743		
PM= 0	10	10	9	8	7	3	2	1
in path:	GC	10,091	10,091	6,511	17,421	103,953	103,953	124,743
PM= 0	12	12	11	7	3	2	1	
in path:	GC	9,170	11,004	17,421	103,953	103,953	124,743	

Table A.9.18

Iteration 18

18		Iteration number							
Paths to change	Variation of the pumping head calculation					Pumping head [m]: 700.10			Remaining $\Delta\Delta h$ m
	φ Mixed	φ after change	$\Delta \Delta h$	Affected paths	PM	φ Mixed	φ	PM	
	mm	mm	m		m		mm	m	
Change		700	-	1	30.9	Completed	700	25.5	1.1
		700	-	1	21.0		700	15.6	
		700	-	1	45.8		700	40.4	
	500	500	5.40	-	-		500	11.7	
	450	400	-	DS	-		450	400	
Change		250	-	DS	-		250	0.0	6.0
		600	11.36	-	-	600	500	47.5	
	450	400	-	DS	-	450	400	35.6	
		350	-	DS	-		350	38.7	
		350	-	DS	-		350	0.0	
		300	-	DS	-		300	4.8	
		350	-	DS	-		350	0.0	
Variation of the Pumping Head					5.40				
The path changes					Partially				

Iteration: 18		Identification of the Critical Paths						
		Increase the size of the pipe, or pump the head loss?:						Increase
		How many Critical Paths?						Several CP
PM= 0	5	5	4	3	2	1		
in path:	GC	18,958	21,757	103,953	103,953	124,743		
PM= 0	6	6	4	3	2	1		
in path:	GC	8,426	21,757	103,953	103,953	124,743		
PM= 0	10	10	9	8	7	3	2	1
in path:	GC	10,091	10,091	6,511	17,421	103,953	103,953	124,743
PM= 0	12	12	11	7	3	2	1	
in path:	GC	27,024	11,004	17,421	103,953	103,953	124,743	

Table A.9.19

Iteration 19

Iteration number									
19									
Paths to change	Variation of the pumping head calculation					Pumping head [m]: 699.05			Remaining $\Delta\Delta h$
	φ Mixed	φ after change	$\Delta \Delta h$	Affected paths	PM	φ Mixed	φ	PM	
	mm	mm	m		m		mm	m	m
		700	-	1	25.5		700	24.4	
		700	-	1	15.6		700	14.5	
		700	-	1	40.4		700	39.3	
		500	-	1	11.7		500	10.6	
Change	450	450	1.06		-	Completed	450	0.0	
Change		300	16.58		-	300	250	0.0	15.5
Change	600	600	5.95		-	600	500	47.5	4.9
	450	400	-	DS	-	450	400	35.6	3.6
		350	-	DS	-		350	38.7	
		350	-	DS	-		350	0.0	
		300	-	DS	-		300	4.8	
		350	-	DS	-		350	0.0	
Variation of the Pumping Head					1.06				
The path changes					Partially				

Iteration: 19		Identification of the Critical Paths						
Increase the size of the pipe, or pump the head loss?:							Increase	
How many Critical Paths?							Several CP	
PM= 0	5	5	4	3	2	1		
in path:	GC	18,958	54,770	103,953	103,953	124,743		
PM= 0	6	6	4	3	2	1		
in path:	GC	8,426	54,770	103,953	103,953	124,743		
PM= 0	10	10	9	8	7	3	2	1
in path:	GC	10,091	10,091	6,511	17,421	103,953	103,953	124,743
PM= 0	12	12	11	7	3	2	1	
in path:	GC	27,024	11,004	17,421	103,953	103,953	124,743	

Table A.9.20

Iteration 20

Iteration number									
20									
Paths to change	Variation of the pumping head calculation					Pumping head [m]: 696.64			Remaining $\Delta\Delta h$
	φ Mixed	φ after change	$\Delta \Delta h$	Affected paths	PM	φ Mixed	φ	PM	
	mm	mm	m		m		mm	m	
Change		700	-	1	24.4		700	22.0	
		700	-	1	14.5		700	12.1	
		700	-	1	39.3		700	36.9	
		500	-	1	10.6		500	8.2	
		500	2.41		-		500	0.0	
	300	300	15.52		-	300	250	0.0	13.1
	600	600	4.90		-	600	500	47.5	2.5
	450	400	-	DS	-	450	400	35.6	3.6
		350	-	DS	-		350	38.7	
		350	-	DS	-		350	0.0	
		300	-	DS	-		300	4.8	
		350	-	DS	-		350	0.0	
Variation of the Pumping Head					2.41				
The path changes					Partially				

Iteration: 20		Identification of the Critical Paths						
Increase the size of the pipe, or pump the head loss?:							Increase	
How many Critical Paths?							Several CP	
PM= 0	5	5	4	3	2	1		
in path:	GC	39,552	54,770	103,953	103,953	124,743		
PM= 0	6	6	4	3	2	1		
in path:	GC	8,426	54,770	103,953	103,953	124,743		
PM= 0	10	10	9	8	7	3	2	1
in path:	GC	10,091	10,091	6,511	17,421	103,953	103,953	124,743
PM= 0	12	12	11	7	3	2	1	
in path:	GC	27,024	11,004	17,421	103,953	103,953	124,743	

Table A.9.21

Annex: Published Articles

Article I:

Title: Pump Efficiency Analysis for Proper Energy Assessment in Optimization of Water Supply Systems.

Authors: Araceli Martin-Candilejo, David Santillán and Luis Garrote.

Journal: Water, MDPI.

Date: Published on the 31st of December 2019.

Article II:

Title: Optimization of the Design of Water Distribution Systems for Variable Pumping Flow Rates.

Authors: Araceli Martin-Candilejo, David Santillán, Ana Iglesias and Luis Garrote.

Journal: Water, MDPI.

Date: Published on the 28th of January 2020

Article

Pump Efficiency Analysis for Proper Energy Assessment in Optimization of Water Supply Systems

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Abstract: Water supply systems need to be designed accounting for both construction and operational costs. When the installation requires water pumping, it is key for the operational costs to know how well the pump can perform. So far, pump efficiency has been considered using conservative values, in the absence of a better estimation. The aim of this paper was to improve determining the energy costs by clarifying what the value of the pump performance should be. For this, 226 commercial pumps were studied, registering the efficiency at the optimum operating point, as well as other variables such as the flow rate, height, and pump type. As a result, a strong relationship between the pump performance and the discharge flow was spotted. That allowed the generation of an empirical curve, which can be used by designers to anticipate what pump efficiency can be expected. The results are used in a simple case study using the Granados Optimization System. These achievements can be implemented in design policies for a better energy assessment in the optimization of water supply systems.

Keywords: pump efficiency; water distribution systems; water supply systems; optimization; design policies; design

1. Introduction

Under the climate change threat, becoming energetically efficient is now a crucial necessity for our society. A proper energy assessment is, therefore, a key factor to account for in the developing of new policies that look after sustainable designs of water supply systems [1]. The designing stage of a water distribution system requires considering not only the construction phase, but also the operation of the facility over its entire lifespan. With this approach, energy expenditure takes a huge fraction of the final cost, and it should not be dismissed from the calculations. There are many designing approaches that do incorporate this aspect in their analysis. Mala-Jetmarova et al. [2] summarized what other authors considered in their proposals. After their review of the state of art, it seems that the design of water distribution systems is increasingly emphasizing the importance of including operation assessments.

The operational costs are often included in optimization algorithms in a single economic function that would also include construction costs of the network. This approach is used in some classical studies, like [3], but it is still the most common way to assess energy costs in the design. That is the case of [4–9] or [10]. Nevertheless, a different perspective is to treat operational and construction costs as separated objectives, as [11] does. However, this assessment confirmed that such a perspective throws back very different design solutions: the optimum design for construction costs means the highest operational investment and vice versa. This approach makes the search of the optimum for the overall installation more difficult, and therefore it is more advisable to adopt a single cost function to optimize.

In order to achieve a more precise assessment of all costs involved, multiobjective algorithms have started to incorporate in the optimization function costs of maintenance [12,13], replacement [14–16], and greenhouse gas emissions [17–20], among others. Multiobjective methods offer a very complete revision of all costs, but many times they require complex programming, resulting in being computationally expensive and hard to use. The work of [21] explained the state of the art of the multiobjective techniques.

Because operation costs are distributed along the lifespan of the installation, they need to be computed at the design stage. The equivalence is calculated using an accumulative factor that depends on the duration of the operation of the water drive considered, the duration of the construction period, and the discount rate employed. Regarding the lifespan of the facility, some methodologies consider an operative life of 20 years [3,5], but other authors [15,16,20] prefer to carry out the analysis for 100 years.

Another key variable for the operation costs is the pump efficiency. One of the most widely used methods for designing water supply systems in Spain is the Granados System [22,23], which is a gradient-based procedure. Among all variables affecting the Granados System, the pump efficiency is the key factor for calculating the diameter of the hydraulic conduction. The aim of this paper is to define the relationship between the pump's efficiency and the other parameters involved in the procedure, such as the flow rate, the pumping head and the required power. This pump efficiency analysis does not only serve for the purposes of the Granados System, but it can also help in many other methods that require the value of the pump's efficiency. This is the case of [24], where they use an estimated value of 75% in their calculations; for Wu et al. [15], the values range from 81% to 84%; Gessler and Walski [25] estimate the efficiency as 75% and so do Alperovits and Shamir [26] and Featherstone and El-Jumaily [27].

Other studies have also decided to optimize the design using as key variables the pump location [28,29], pump capacity [4], type [19], power [3], pumping head [30–32], pumping schedule [24,33], or pressure [34,35]. Water supply system design is a complex task where many variables are involved [36] and inter-connected; the decision-making needs a full comprehension of how each factor affects the installation. Some of the relationships between variables are still unknown and only intuited by experience. For this purpose, sensitivity analyses are necessary. The work [37] carries out an exhaustive sensitivity analysis using Sobol's method (a variance-based approach) to determine the variables that most affect the installation. These variables could vary from pipe diameters to tank sizes, and the degree of influence depends on each case study. In their work, they prove the computational savings that can be obtained and, therefore, how beneficial the analysis is. This paper also intends to serve as a sensitivity analysis of how much pump efficiency is influenced by other factors and how the pump's efficiency can affect the final cost of a water supply system.

Since the pump efficiency can substantially affect the design of water supply systems due to the wide range of values they can adopt, with this study we carry out an extensive study of the pump performance in order to characterize its values. For this to be made, we analyze the features of commercial pumps available in the market. The conclusions are integrated in a gradient-based procedure, using as a base the Granados System, giving rise to an optimized design method that accounts for the construction and operational costs. This procedure is also compatible with adding other variables such as the carbon footprint [38] of the design or the GHG emissions. Our new methodology is applied to a case study to illustrate its computationally straightforward conception.

2. Materials and Methods

2.1. Brief Summary of the Granados System

Granados System is a pipe-sizing method indicated for the design of branched water distribution systems. It is a gradient based methodology. It is, in fact, based on the 'change gradient' concept: The change gradient is defined as the cost of reducing one meter of head loss by increasing the pipe diameter from ϕ_i to the next bigger one $\phi_j = \phi_{i+1}$:

$$GC_{\phi_i \rightarrow \phi_j}^q = \frac{P_j - P_i}{\Delta h_i^q - \Delta h_j^q}, \quad (1)$$

where P_i and P_j are prices of pipes of length L and diameters ϕ_i and ϕ_j , respectively; Δh_i^q and Δh_j^q are the head losses of a pipe of length L and diameters ϕ_i and ϕ_j , respectively, when a flow rate q is circulating. When the head losses are calculated using Manning's formulation, the change gradient's expression is:

$$GC_{\phi_i \rightarrow \phi_j}^q = \frac{\frac{P_j L - P_i L}{\frac{L n_i^2 2^{20/3} q^2}{\pi^2 \phi_i^{16/3}} - \frac{L n_j^2 2^{20/3} q^2}{\pi^2 \phi_j^{16/3}}} = \frac{\pi^2}{n^2 2^{20/3}} \frac{(p_j - p_i)}{\left(\frac{n_i}{\phi_i^{16/3}} - \frac{n_j}{\phi_j^{16/3}} \right)} \frac{1}{q^2} = K_{GC} \frac{1}{q^2}, \quad (2)$$

with n being Manning's friction coefficient.

The definition of the change gradient also needs to include some other associated costs such as excavation expenditure or reinforcements against chemically aggressive environment or hydraulic transients. During the optimization stage, hydraulic transients are usually taken into account in a simplified way, for instance, extra thickness of the pipes to withstand water hammer transient pressure. Once the optimum design is achieved, proper assessment of water hammer should be carried out to verify the initial hypothesis. These associated cost factors increase the pipe price, and therefore, they have to be accounted in the design through P_i and P_j .

Increasing the pipe's diameter means a reduction in the head loss along the pipeline, but it has the extra cost of the wider and more expensive tube. Since the change gradient is the cost of reducing the head loss by one meter, it needs to be compared to the cost of the energy required for pumping the water at one meter height throughout the entire life of the facility, C_{E1} , which reads:

$$C_{E1} = C_{aE1} \cdot f_A = f_A 9,81 \frac{V}{3600} \frac{1}{\mu_B} \frac{1}{\mu_M} p_E, \quad (3)$$

$$f_A = \frac{(1+i)^{n_U} - 1}{(1+i)^{n_U} i} \cdot \frac{1}{(1+i)^{n_C}}, \quad (4)$$

where C_{aE1} is the annual energy cost per meter, f_A is the discount factor, n_C is the duration of the construction period, n_U is the useful life of the installation, i is the discount rate, V is the annual volume of water to pump, μ_B and μ_M are the pump and engine efficiency, and lastly p_E is the unit price of energy.

As it is justified in Granados' work, with the exception of the pump performance μ_B , the other variables in the previous equation are relatively known data: V is the total demanded volume to pump, p_E is the unit price of the energy that has been hired, and f_A is calculated from the discount rate i , the service life n_U of the water pipeline which depends on the chosen material, and the construction period n_C . Regarding the engine efficiency μ_M , although it varies theoretically depending on the model chosen and the operating point of the pump, the variations in engine performance are so small that it can be considered constant across different models and manufacturers. Therefore, the above equation can be simplified to:

$$C_{E1} = K C_{E1} \frac{1}{\mu_B}. \quad (5)$$

As previously said, Granados System consists of comparing the cost of building a wider pipe (change gradient) and reducing the head loss by a meter, to the cost of pumping one meter of head loss (C_{E1}). The reasoning is the following:

- If $GC_{\phi_i \rightarrow \phi_{i+1}}^q < C_{E1} \Rightarrow$ Diameter ϕ_{i+1} is preferable to ϕ_i .
- If $GC_{\phi_i \rightarrow \phi_{i+1}}^q > C_{E1} \Rightarrow$ Diameter ϕ_i is preferable to ϕ_{i+1} .

But since C_{E1} depends on the pump efficiency μ_B , the procedure changes to:

- If $GC_{\phi_i \rightarrow \phi_{i+1}}^q < C_{E1} \Rightarrow GC_{\phi_i \rightarrow \phi_{i+1}}^q < K C_{E1} \frac{1}{\mu_B} \Rightarrow \mu_B < \frac{K C_{E1}}{GC_{\phi_i \rightarrow \phi_{i+1}}^q} \Rightarrow$ Move to ϕ_{i+1} .
- If $GC_{\phi_i \rightarrow \phi_{i+1}}^q > C_{E1} \Rightarrow GC_{\phi_i \rightarrow \phi_{i+1}}^q > K C_{E1} \frac{1}{\mu_B} \Rightarrow \mu_B > \frac{K C_{E1}}{GC_{\phi_i \rightarrow \phi_{i+1}}^q} \Rightarrow$ Keep ϕ_i .

This means that, whenever there is a pump on the market whose efficiency can be greater than the calculated μ_B , the optimum diameter will be ϕ_i . In the event that no commercial pump can reach that performance because it is very high, it will be necessary to move to the next diameter ϕ_{i+1} . Therefore, to apply this method it is necessary to know the maximum efficiency that pumps can reach. This is an uncertainty of the Granados System, and up to now, typically, pump efficiency values around 80% are already indicative.

2.2. Methodology

Among all variables affecting Granados System, the pump performance is the key factor for calculating the diameter of the hydraulic conduction. Nevertheless the results may vary significantly depending on the pump efficiency; indeed, μ_B presents a wide range of possible values that typically goes from 70% up to 90%. This means a variability of almost 20% in the estimation of the cost, which is a substantial difference. Therefore the aim of this paper is to define the relationship between the pump's performance and the other parameters involved in the procedure, such as the flow rate, the pumping head or the required power. For this to be done, we select 400 commercial pumps from the catalogues of several manufactures. We discard custom-made pumps since the cost of these pumps is much higher than the ordinary ones listed and the offering in commercial catalogues is already very wide. The selected pumps vary from each other in their type, impeller diameter, number of stages, rotation speed (electrical current frequency, number of poles), brand, etc. For all these models, we study the pump efficiency; in particular, we register the optimum value together with the correspondent flow rate, head and power consumed. Nevertheless, some pumps are ruled out of the sample and presentation of the results because they either were similar to those of other manufacturers, or because they were very specific for some industrial or sanitary engineering uses. In the end, the sample consists of 226 hydraulic pumps.

For the detailed study of the pump performance, this research has focused on the most common type of pump for the applications in civil engineering (supply, irrigation, sanitation, etc.)—centrifugal pumps. Within centrifugal pumps, both horizontal and vertical axes are selected, mostly with a radial flow configuration, with the exception of submersible pumps, for which the axial arrangement is more common. The sample includes regular horizontal and vertical pumps, split case pumps, multistage and submersible pumps. The manufacturers used for this analysis were IDEAL, WILO, ESPA and HASA. More specifically, the commercial models were:

- Split case pumps: CP/CPI/CPR series.
- Horizontal pumps (normalized in the European Union): RNI/RN series.
- Multistage horizontal pumps: APM series.
- Vertical pumps: VS/VG series.
- Submersible vertical pumps: SVA/SVH series.

Multistage pumps perform with the same efficiency for a specific flow rate and different heights (which is the number of stages multiplied for the unitary head). To avoid this dispersion that could make it difficult to draw conclusions, it was decided to only use the optimum operating point correspondent to a single stage.

Using the data collected for the optimum operating points for all the 226 hydraulic pumps previously mentioned (pump's optimum efficiency and associated flow rate, head, speed, power, frequency, diameter, etc.) we carried out an analysis to establish relationships among the design variables of a water drive.

3. Results and Discussion

Figure 1a shows the values of the optimum efficiency of the pump and the flow rate correspondent to that point. From it, it can be seen that there is a relationship between both variables: the performance of the pump improves as the discharge increases, although it seems to reach a horizontal asymptote for μ_B around 90%. In this same figure, it can be spotted that the relationship between the two variables is more precise for flow rate values that are greater than 500 L/s. Below this value, as it can be seen in

Figure 1b, there is more dispersion. Also, when the discharge is at least 100 L/s, the pump efficiency can be expected to be better than 80%, but under it, the dispersion is stronger. On the other hand, it has been studied whether the type of pump has any relation with the efficiency. For that, Figure 1 also shows the distribution of the operating points for the split case pumps, horizontal and multistage pumps, horizontal and multistage (both horizontal and vertical).

(a)

(b)

Figure 1. (a). Optimum pump efficiency and the correspondent flow rate at that operating point, classified by the pump type. (b) Detail of the previous figure for smaller discharges.

Regarding the pump type, we conclude that:

- Split case pumps are, in general, the ones that provide the best performance for flow rates over 500 L/s.

- Between 100 L/s and 500 L/s, both split case and horizontal offer the best results.
- Under 100 L/s, the distribution is very heterogeneous and disperse, but as a general rule, there is always a horizontal pump that can provide the best efficiency (but also the lowest, due to the dispersion). Vertical pumps also show great performances, but they can generally be exceeded by a horizontal model.
- Vertical multistage pumps always give a better efficiency than horizontal multistage pumps.
- For small flow rates, the differences between the pump efficiency among models are very high. Different models can almost double other pump's efficiency, meaning that the energy cost can be almost twice as much if the pump is not well selected. This is why the pump analysis is more complicated for small discharge values.

When the pumping height and the efficiency are plotted together, as shown in Figure 2, there is no clear sign of a relationship between the two variables since the dispersion is too strong for any head value. There is a very light tendency of high pump performances (around 80%) for under 40 m head, but over 60 m the distribution of μ_B is too scattered.

Figure 2. Optimum pump efficiency and the correspondent pumping height at that operating point.

To test out what the relationship between the optimum μ_B and the required power for that operating point is, Figure 3 was elaborated.

The main conclusions drawn from Figure 3 are:

- There is a relationship between the pump efficiency and the required power for the optimum operating point.
- The relation seems to be clearer from 400 kW and on, and values greater than 80% efficiency can be expected.
- This relationship seems to be weaker than the existing one between the flow rate and the pump efficiency. The interpreted reason for this fact is that, since the power is calculated from the flow rate and the pumping height, the good relationship that the flow rate transfers to the power is weakened by the poor bond that the pumping head and efficiency have. The authors of [3] had already presented similar conclusions. In their work, they empirically proved that the cost of pumping depended on power; which indeed agrees with these results.

(a)

(b)

Figure 3. (a) Optimum pump efficiency and the correspondent pump power at that operating point. (b) Detail of the previous figure.

To summarize, this first assessment concludes that the strongest of the relationships with the pump efficiency is that of the flow rate, especially when the flow is greater than 100 L/s and it is almost linear.

On the other hand, the relationship between the specific speed of the pump and the efficiency is shown in Figure 4. The specific speed of the pump is defined as:

$$n_s = \frac{n_r \cdot q^{1/2}}{H^{3/4}}, \quad (6)$$

where n_s is the specific speed; q , H and n_r are the flow rate, pumping head and the rotation speed, respectively, at the optimum operating point [39]. It is interpreted as the rotation speed that a geometrically similar pump should have in order to elevate a discharge of $1 \text{ m}^3/\text{s}$ at 1 m height.

Figure 4. Optimum pump efficiency and the correspondent specific speed of the pump, classified by the pump type.

Low specific speed indicates that the pump is suitable for small flow rates and great heights. This means that, since small flow rates have a poorer relationship with the pump efficiency, pumps with a low specific speed will have the worst performance values. This is shown in Figure 4. On the contrary, high specific speed is an indicator of a pump suitable for great flow rates and low heights. According to the previous analysis made from Figure 1, greater discharges correspond to better performances, and therefore, a high specific speed can be associated with a good μ_B , as Figure 4 shows. The scattering of the figure is caused by the poor relationship that the pumping height and the rotation speed have with the pump efficiency. However there is a diffuse tendency of increasing μ_B as the specific speed grows. As far as pump type is concerned, the types of pumps that operate at higher flow rates will be the ones with a greater specific speed and therefore, better performance: These are split case pumps, as shown in Figure 4. Vertical multistage pumps also give good results in the smaller flow rate ranges.

3.1. Relationship between the Flow Rate and the Pump Efficiency

From the previous figures, it seems that there are three flow rate ranges where the relationship with the pump efficiency is different. The first one would be from 0 to 100 L/s , the second from 100 to 500 L/s and the last one over 500 L/s . Nevertheless, in order to be more precise, instead of three zones, the curve has been fitted using up to fourteen flow rate subdivisions. Table 1 shows the average optimum pump efficiency for each interval, as well as the maximum and minimum found among the studied pumps.

Table 1. Elaboration of the adjusted curves of the pump efficiency versus the flow rate.

Interval Number	Pump Efficiency for Each Flow Rate Interval					
	Flow Rate Range			Pump Efficiency		
	Minimum q (L/s)	Maximum q (L/s)	Average q (L/s)	Average μ_B (%)	Maximum μ_B (%)	Minimum μ_B (%)
1	0	5	2.5	55.3%	77.0%	36.2%
2	5	10	7.5	63.5%	76.0%	45.0%
3	10	15	12.5	65.7%	77.0%	45.0%
4	15	25	20	74.2%	82.0%	63.0%
5	25	40	32.5	75.0%	85.0%	61.2%
6	40	65	52.5	78.6%	84.0%	70.5%
7	65	100	82.5	81.0%	85.4%	75.4%
8	100	150	125	83.1%	86.0%	74.0%
9	150	200	175	82.8%	87.0%	80.4%
10	200	300	250	83.9%	88.0%	81.0%
11	300	500	400	84.9%	88.8%	79.9%
12	500	1000	750	85.5%	88.5%	80.4%
13	1000	2000	1500	88.3%	89.0%	86.7%
14	2000	3000	2500	88.2%	89.0%	86.2%

When these values are adjusted through a doubly logarithmic curve, the relationships for both the average and maximum values fit satisfactorily ($r^2 > 98\%$ and $r^2 > 90\%$, respectively), as can be seen in Figure 5. The empirical equations that relate the optimum pump efficiency and the flow rate are:

$$\mu_B^{\text{Average}} = 0.1286 \ln (2.047 \ln q - 1.7951) + 0.5471 \quad r^2 > 98\%, \quad (7)$$

$$\mu_B^{\text{Maximum}} = 0.0576 \ln (2.047 \ln q - 1.7951) + 0.741 \quad r^2 > 90\%, \quad (8)$$

where q is the flow rate in liters per second (L/s).

Figure 5. Adjusted curves. The curves are elaborated using 14 intervals of the collected data.

Finally, Figure 6 shows the adjusted curves along with the collected data. Both curves fit satisfactorily for most flow rate ranges, especially for the bigger ones; nevertheless the dispersion is higher for the small discharge values (under 50 L/s), but still, it gives a valid reference. To facilitate the visualization, Figure 6b shows the results in logarithmic scale.

(a)

(b)

Figure 6. (a) Optimum pump efficiency curve: database and mathematical adjustment; (b) mathematical adjustment in logarithmic scale.

3.2. Application of the Pump Efficiency Curves to the Granados System

For a better comprehension, the full procedure is shown in Figure 7. On the figure, all variables affecting the change gradient and the energy cost are graphically represented: flow rate, pump efficiency, energy price, annual volume of water, engine efficiency, useful life of the installation, construction period, discount rate, commercial diameters, pipe prices and pipe roughness.

Figure 7. Pipe sizing methodology representing the steps for the design and the shapes that curves might present depending on the variables.

The design process represented in Figure 7 consists of the following steps:

1. Calculate the design demand flow rate of the hydraulic drive.
2. Calculate the change gradient curves for a commercial series of diameters with Equation (2). Since the aim is to compare the construction costs to those of the energy, excavation costs also must be considered and added to these figures.
3. Calculate the expected pump efficiency using the average pump efficiency equation (Equation (7)). It is always more conservative to use the average pump efficiency equation rather than the maximum one, nevertheless this one can also be used but it would require a much more exhaustive pump search.
4. Calculate the annual energy cost for one meter height Ca_{E1} using Equation (3).
5. At this point, different alternatives might be considered for various energy prices, etc.
6. Calculate the total accumulated energy cost for one meter height C_{E1} with Equations (3) and (4).
7. Compare the change gradient and the energy cost C_{E1} and select the pipe diameter: when $GC_{\phi_i \rightarrow \phi_{i+1}}^q < C_{E1}$ select the wider diameter ϕ_{i+1} , but when $GC_{\phi_i \rightarrow \phi_{i+1}}^q > C_{E1}$ select diameter ϕ_i . When $GC_{\phi_i \rightarrow \phi_{i+1}}^q = C_{E1}$, it is always preferable to build a bigger diameter ϕ_{i+1} , in case energy price, discount rate, etc. change.

3.3. Case Study: Navas del Marqués

To exemplify the application of the previous design procedure, the method will be used in a case study. It is based on the dam project in the Navas del Marqués locality, Ávila, Spain, which was carried out by the Tagus Hydrographic Confederation [40]. In this section we intend to compare the design results obtained in the project to the ones that would be obtained with the proposed procedure. The project included the following works:

- New dam.
- New pumping station.

- New pipe replacing the first part of the water drive. There was a tunnel section from the previous water supply system that will remain as it was.
- New raw water deposit, previous to the water treatment station.
- New regulation deposit, after the water treatment station.
- New distribution network.

Figure 8 shows the schematic works that the project included, along with some relevant altitudes, and input data. This paper is concerned about the sizing of first part of the water drive, i.e., Pipe 1 (remember that the tunnel section, Pipe 2, was to remain in the original state).

Figure 8. Case study at Navas del Marqués.: Pipe 1 is the only one to replace.

The project data that are used can be summarized in the following list:

- Design flow rate: 0.164 m³/s.
- Annual volume of water: 1.5 hm³/year.
- Pipe diameters and the correspondent prices. These are compiled in Table 2.
- Energy price: 0.072 €/kWh.
- Duration of the construction period: 30 months.
- Although it is not specified in the project, pump engine efficiency is fixed at 94%.

This design process can be also followed throughout all series of Figure 9. These figures follow the same color code and symbology as Figure 7. The calculation followed the steps listed above:

1. The design discharge flow is the one used by the original project which is $q = 0.164$ m³/s.
2. For the calculation of the change gradients, the material for the pipe is ductile cast iron. Two different studies have been elaborated for two different pipe prices. Firstly, the pipe prices used in the project have been considered. Secondly, we used the pipe prices proposed by the Canal de Isabel II. Canal de Isabel II is the public authority in charge of the integral water supply of Madrid, Spain, and they published the average pipe prices they use in their infrastructure. Since the Tagus Hydrographic Confederation is also a public company, these prices have also been considered for a sensitivity analysis. On the other hand, it must be kept in mind that the prices used for the change gradient include that of the full construction cost (including excavation, joining, etc.). Table 2 shows the pipe prices and the change gradients.

Figure 9. Case study of the Navas del Marqués. (a) Step 4: Calculation of the annual energy cost; (b) Step 5: Calculation of the capitalized energy cost throughout the entire life of the installation. (c,d) Both panels show steps 1 and 5 of the procedure. This means that both the change gradient calculation (red curves) and the pipe selection is shown in the figures. In (c), the change gradients are calculated using the Canal de Isabel II commercial diameters, and in (d), they used the project data. The big orange circle in (d) represents the main solution, whilst the small orange circles represent the solutions for the other alternatives evaluated in the case study. For the main solution, as the energy cost per meter CE_1 is greater than $CG_{Ø350 \rightarrow Ø400}$, it is more convenient to choose Ø400 mm. But because CE_1

is smaller than CGØ400→Ø450, it should not be passed onto Ø450 mm and remain with Ø400 mm. The same reasoning applies to the other solution alternatives.

Table 2. Pipe prices series and change gradients calculations.

Pipe Diameter	Change Gradients			
	Canal de Isabel II		Project Diameters	
	Construction Cost	Change Gradient	Construction Cost	Change Gradient
mm	€/m	€/m	€/m	€/m
60	55	0.11		
80	64	0.26		
100	69	2.11		
125	79	4.84		
150	86	30	103 ¹	36
200	105	233	127	256
250	134	796	159	1044
300	161	3070	194	2373
350	197	6670	221	6593
400	227	15,972	252	11,226
450	260	33,781	295	64,309
500	295	98,565	341	76,920
600	378		405	

¹ The project includes a limited series of ductile cast iron pipes in the estimated budget documents. These start from 150 mm.

- Using Equations (7) and (8), the average pump efficiency obtained is 82.44% and the maximum value is 86.52%. This same result can be obtained from Figure 6a with the design flow rate $q = 0.164 \text{ m}^3/\text{s}$.
- The calculation of the annual energy cost per meter has been made for different values of the energy price. Engine efficiency was fixed at 94% and the demanded annual volume of the project is 1.5 hm^3 . Energy prices used for the calculations are 0.072 €/kWh, which, using Equation (3), gives an annual energy cost of 380 €/m/year; and also 0.125 €/kWh, for which Ca_{EI} is 659 €/m/year. This will allow a sensibility analysis to see how robust the design is against energy price changes. The first price, 0.072 €/kWh, corresponds to the energy price used in the project, and the second alternative, 0.125 €/kWh, corresponds to a higher energy price. These results can be seen in Figure 9a.
- Once the annual unit pumping cost is obtained, the total accumulated cost is calculated using Equation (3). We assume a life span for the cast iron pipe of 31 years. Likewise, the duration of the expected construction period according to the project is 30 months, which corresponds to 2.5 years. We analyzed the sensibility of the design for the discount rate. In this line we adopt three values of the flow rate, where i equals 3%, 4% and 5%. With all this, the results obtained in each situation of energy prices and discount rate are shown in Table 3 below and in Figure 9b.

Table 3. Selection of the pipe diameter, depending on the energy prices, diameter series and discount rate considered.

Pipe Selection						
Annual Unit Energy Cost		Total Unit Energy Cost			Selected Diameter	
Energy Price	Ca_{EI} €/m/year	i %	f_a	C_{EI} €/m	Canal Isabel II Pipes mm	Project Pipes mm
0.125 €/kWh	659	3	18.6	12,248	(a) 400 *	(g) 450
		4	15.9	10,513	(b) 400	(h) 400
		5	13.8	9100	(c) 400	(i) 400
0.072 €/kWh	380	3	18.6	7055	(d) 400	(j) 400
		4	15.9	6056	(e) 350	(k) 350
		5	13.8	5242	(f) 350	(l) 350

* The letters in brackets indicate the graphical solutions in Figure 9 (c) and (d).

6. Once the full energy cost has been obtained, as well as the change gradients, the comparison is made, following the reasoning previously described. The selected pipes are compiled in Table 3 and shown in Figure 9c,d.

As it is shown, most of the alternatives throw back a pipe diameter solution of 400 mm; this is not too far from the solution taken in the project, which is 500 mm. The diameter selected in the project implies a facility with higher costs than the design obtained with the optimization of this research. However the project solution is energetically less consuming, and this is a positive fact because energy prices tend to increase with time, leading to significant increases in the energy cost. Nevertheless, when the sizing is made for project energy prices, the solutions oscillate between 350 and 400 mm. Although the project and presented method give close results, the new methodology can help to optimize the full cost of the facility.

An economic evaluation has been conducted using the project energy prices and pipe prices. This can be seen in Table 4 and Figure 10.

Figure 10. Economic evaluation of the case study at the Navas del Marqués.

Table 4. Economic evaluation for the case study using the project prices for the pipes, once a diameter of 400 mm has been selected, to prove the least cost result.

Diameter Ø (mm)	Economic Evaluation				Total Cost (€)
	Construction Costs €	Head Loss	Annual Cost	$i = 3\%$	
		Δh (m)	caE (€/year)	CE (€)	
300	329,886	35.01	28,374	527,076	856,962
350	376,444	15.39	20,922	388,646	765,091
400	428,121	7.55	17,946	333,353	761,474
450	501,646	4.03	16,609	308,516	810,162
500	578,985	2.30	15,951	296,302	875,288

Table 4 and Figure 10 show how diameters Ø400 mm and Ø350 mm give very close results for the total cost, since they only differ in approximately 4000 €. Nevertheless, the cost variation increases compared to diameter Ø500 mm and it is significantly higher, as it can be seen in Figure 10 even though the results only differ in 100 mm wide. When the sizing is made for higher energy prices, the

most common result is 400 mm. As a conclusion, although the experience of the designer can never be replaced, this method is a very convenient tool to find the optimum diameter of a pipe.

4. Conclusions

The present pump analysis is a useful tool to help with the uncertainties regarding decision-making in water supply system design. Because the relationship of the pump efficiency with the other variables involved in the design process has been elucidated, the understanding of such a complex procedure as the design of a water supply system is improved.

The definition of the pump-efficiency–flow-rate curve reduces the conflict resolution involved in gradient-based methods, avoiding iterations regarding the pump efficiency. This automatically leads to less computational effort. In this sense, the Granados method is a straightforward procedure.

Some current methods are somewhat black-boxes, losing part of their utility by not being easy to comprehend for design engineers. The visual representation of our design methodology presented in this paper facilitates the understanding of the influence of each variable and allows to have a clear picture of the process. This simplicity is one of the main necessities to aim for when it comes to water supply system design.

As the case study shows, energy costs are proved to be a great fraction of the full cost of the installation, and they need to be considered from the first phase of the design. In contrast with other methodologies that do not consider these energy cost, this analysis aims for an integrated water assessment where all costs of the water–energy nexus are integrated.

Here, we obtain empirical equations that define the average and maximum optimal pump efficiency to expect depending on the flow rate of the installation. These results can be incorporated to elaborate a proper energy assessment of the operational costs. As the case study has shown, the design of a water supply system is also very much influenced by the construction costs (the design differs when Canal de Isabel II or project pipe prices are used) and by the energy rates (centesimal order variations will throw back different results). This reinforces the importance of carrying out a sensitivity analysis when it comes to designing a water supply system. These conclusions can be used for policy assessment in integrated water management as well as supply system design.

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References

1. Sanchis, R.; Díaz-Madroñero, M.; López-Jiménez, P.A.; Pérez-Sánchez, M. Solution Approaches for the Management of the Water Resources in Irrigation Water Systems with Fuzzy Costs. *Water* **2019**, *11*, 2432.
2. Mala-Jetmarova, H.; Sultanova, N.; Savic, D. Lost in optimisation of water distribution systems? A literature review of system operation. *Environ. Model. Softw.* **2017**, *93*, 209–254.
3. Kim, J.H.; Mays, L.W. Optimal rehabilitation model for water-distribution systems. *J. Water Resour. Plan. Manag.* **1994**, *120*, 674–692.
4. Kang, D.; Lansey, K. Scenario-based robust optimization of regional water and wastewater infrastructure. *J. Water Resour. Plan. Manag.* **2012**, *139*, 325–338.
5. Dzedzic, R.; Karney, B.W. Cost gradient-based assessment and design improvement technique for water distribution networks with varying loads. *J. Water Resour. Plan. Manag.* **2015**, *142*, 04015043.
6. Samani, H.M.; Mottaghi, A. Optimization of water distribution networks using integer linear programming. *J. Hydraul. Eng.* **2006**, *132*, 501–509.
7. Costa, A.; De Medeiros, J.; Pessoa, F. Optimization of pipe networks including pumps by simulated annealing. *Braz. J. Chem. Eng.* **2000**, *17*, 887–896.
8. Vamvakiridou-Lyroudia, L.; Walters, G.; Savic, D. Fuzzy multiobjective optimization of water distribution networks. *J. Water Resour. Plan. Manag.* **2005**, *131*, 467–476.

9. Spiliotis, M.; Tsakiris, G. Minimum cost irrigation network design using interactive fuzzy integer programming. *J. Irrig. Drain. Eng.* **2007**, *133*, 242–248.
10. Jin, X.; Zhang, J.; Gao, J.-l.; Wu, W.-y. Multi-objective optimization of water supply network rehabilitation with non-dominated sorting genetic algorithm-II. *J. Zhejiang Univ. SCI. A* **2008**, *9*, 391–400.
11. Roshani, E.; Filion, Y. Event-based approach to optimize the timing of water main rehabilitation with asset management strategies. *J. Water Resour. Plan. Manag.* **2013**, *140*, 04014004.
12. Perelman, L.; Ostfeld, A. An adaptive heuristic cross-entropy algorithm for optimal design of water distribution systems. *Eng. Optim.* **2007**, *39*, 413–428.
13. Perelman, L.; Ostfeld, A.; Salomons, E. Cross entropy multiobjective optimization for water distribution systems design. *Water Resour. Res.* **2008**, *44*, doi:10.1029/2007WR006248.
14. Wu, W.; Simpson, A.R.; Maier, H.R. Multi-objective genetic algorithm optimisation of water distribution systems accounting for sustainability. *Proc. Water Down Under* **2008**, *2008*, 1750.
15. Wu, W.; Simpson, A.R.; Maier, H.R. Accounting for greenhouse gas emissions in multiobjective genetic algorithm optimization of water distribution systems. *J. Water Resour. Plan. Manag.* **2009**, *136*, 146–155.
16. Wu, W.; Maier, H.R.; Simpson, A.R. Single-objective versus multiobjective optimization of water distribution systems accounting for greenhouse gas emissions by carbon pricing. *J. Water Resour. Plan. Manag.* **2009**, *136*, 555–565.
17. Wu, W.; Simpson, A.R.; Maier, H.R. Sensitivity of optimal tradeoffs between cost and greenhouse gas emissions for water distribution systems to electricity tariff and generation. *J. Water Resour. Plan. Manag.* **2011**, *138*, 182–186.
18. Wu, W.; Simpson, A.R.; Maier, H.R.; Marchi, A. Incorporation of variable-speed pumping in multiobjective genetic algorithm optimization of the design of water transmission systems. *J. Water Resour. Plan. Manag.* **2011**, *138*, 543–552.
19. Stokes, C.S.; Simpson, A.R.; Maier, H.R. A computational software tool for the minimization of costs and greenhouse gas emissions associated with water distribution systems. *Environ. Model. Softw.* **2015**, *69*, 452–467.
20. Stokes, C.S.; Maier, H.R.; Simpson, A.R. Effect of storage tank size on the minimization of water distribution system cost and greenhouse gas emissions while considering time-dependent emissions factors. *J. Water Resour. Plan. Manag.* **2015**, *142*, 04015052.
21. Reed, P.M.; Hadka, D.; Herman, J.D.; Kasprzyk, J.R.; Kollat, J.B. Evolutionary multiobjective optimization in water resources: The past, present, and future. *Adv. Water Resour.* **2013**, *51*, 438–456.
22. Carrasco, F.J.M.; de Marcos, L.G. *Dimensionamiento y Optimización de Obras Hidráulicas*, 4th ed; Garceta Grupo Editorial: Malaga, Spain, 2013. (In Spanish)
23. Granados, A. *Problemas de Obras Hidráulicas*, Escuela de Ingenieros de Caminos, Canales y Puertos. Servicio de Publicaciones: Madrid, Spain, 1995. (In Spanish)
24. Ostfeld, A. Optimal design and operation of multiquality networks under unsteady conditions. *J. Water Resour. Plan. Manag.* **2005**, *131*, 116–124.
25. Gessler, J.; Walski, T.M. *Water Distribution System Optimization*; Army Engineer Waterways Experiment Station Vicksburg Ms Environmental Lab: Washington, DC, USA, 1985.
26. Alperovits, E.; Shamir, U. Design of optimal water distribution systems. *Water Resour. Res.* **1977**, *13*, 885–900.
27. Featherstone, R.E.; El-Jumaily, K.K. Optimal diameter selection for pipe networks. *J. Hydraul. Eng.* **1983**, *109*, 221–234.
28. Dandy, G.; Hewitson, C. Optimizing hydraulics and water quality in water distribution networks using genetic algorithms. In *Building Partnerships, Joint Conference on Water Resource Engineering and Water Resources Planning and Management 2000, Minneapolis, MN, USA, 30 July–2 August 2000*; Hotchkiss, R.H.; Glade, M., Eds.; ASCE: Reston, VA, USA, 2000; pp. 1–10.
29. Tsakiris, G.; Spiliotis, M. Fuzzy linear programming for problems of water allocation under uncertainty. *Eur. Water* **2004**, *7*, 25–37.
30. Gonçalves, G.M.; Gouveia, L.; Pato, M.V. An improved decomposition-based heuristic to design a water distribution network for an irrigation system. *Ann. Oper. Res.* **2014**, *219*, 141–167.
31. Marques, J.; Cunha, M.; Savić, D. Using real options in the optimal design of water distribution networks. *J. Water Resour. Plan. Manag.* **2014**, *141*, 04014052.

32. Schwartz, R.; Housh, M.; Ostfeld, A. Least-cost robust design optimization of water distribution systems under multiple loading. *J. Water Resour. Plan. Manag.* **2016**, *142*, 04016031.
33. Ostfeld, A.; Tubaltzev, A. Ant colony optimization for least-cost design and operation of pumping water distribution systems. *J. Water Resour. Plan. Manag.* **2008**, *134*, 107–118.
34. Vicente, D.; Garrote, L.; Sánchez, R.; Santillán, D. Pressure management in water distribution systems: Current status, proposals, and future trends. *J. Water Resour. Plan. Manag.* **2015**, *142*, 04015061.
35. Capelo, B.; Pérez-Sánchez, M.; Fernandes, J.F.; Ramos, H.M.; López-Jiménez, P.A.; Branco, P.C. Electrical behaviour of the pump working as turbine in off grid operation. *Appl. Energy* **2017**, *208*, 302–311.
36. Tsakiris, G.; Spiliotis, M. Uncertainty in the analysis of urban water supply and distribution systems. *J. Hydroinform.* **2017**, *19*, 823–837.
37. Fu, G.; Kapelan, Z.; Reed, P. Reducing the complexity of multiobjective water distribution system optimization through global sensitivity analysis. *J. Water Resour. Plan. Manag.* **2011**, *138*, 196–207.
38. Pérez-Sánchez, M.; Sánchez-Romero, F.-J.; López-Jiménez, P.A. Huella energética del agua en función de los patrones de consumo en redes de distribución. *Ingeniería del Agua* **2017**, *21*, 197–212.
39. Pérez-Sánchez, M.; López-Jiménez, P.A.; Ramos, H.M. Modified affinity laws in hydraulic machines towards the best efficiency line. *Water Resour. Manag.* **2018**, *32*, 829–844.
40. Cabrerós, J.M. *Proyecto de Presa Navas del Marqués*; Tajo, C.H.d., Ed.; Ministerio de Medio Ambiente, Ávila, Spain, 1999. (In Spanish)

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Article

Optimization of the Design of Water Distribution Systems for Variable Pumping Flow Rates

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Abstract: Water supply systems need to be designed in an efficient way, accounting for both construction costs and operational energy expenditures when pumping is required. Since water demand varies depending on the moment's necessities, especially when it comes to agricultural purposes, water supply systems should also be designed to adequately handle this. This paper presents a straightforward design methodology that using a constant flow rate, the total cost is equivalent to that of the variable demand flow. The methodology is based on the Granados System, which is a very intuitive and practical gradient based procedure. To adapt it to seasonal demand, the concepts of Equivalent Flow Rate and Equivalent Volume are presented and applied in a simple case study. These concepts are computationally straightforward and facilitate the design process of hydraulic drives under demand variability and can be used in multiple methodologies, aside from the Granados System. The Equivalent Flow Rate and Equivalent Volume offer a solution to design procedures that require a constant flow regime, adapting them to more realistic design situations and therefore widening their practical scope.

Keywords: water distribution systems; optimization; design; pump operation; demand variability

1. Introduction

In September 2015, the United Nations published the 2030 Agenda for Sustainable Development [1], intending to eradicate poverty in all its forms by setting up 17 goals and 169 targets to foster sustainable development through economic, social, and environmental aspects. Goal 6—Water Availability—aims to achieve universal access to drinkable water and sanitation. This Goal is very much related to Goal 7—Sustainable Energy—that aims to ensure sustainable energy for all. In order to achieve the challenges of water and energy, improving the design of hydraulic system is crucial, to ensure efficiency in the use of the resources. The areas in most need of action have less developed water resources systems and scarce economic resources. In these areas, the design strategies need to account for construction costs and the operation of the system, especially when pumping is required, linking energy to water. Design methodologies of water supply systems need to be practical and intuitive for practitioners, and at the same time, reliable and affordable for the users.

Mala-Jetmarova et al. [2] provide an excellent summary of the state of the art in water distribution designs, providing a clear classification of the optimization methods according to the objectives of the optimization (e.g., single objective, multi objective, and others) or the calculation method (e.g., stochastic, heuristic, and others). In more than 70% of the documented choices, the

single objective “least-cost” is selected, and many design proposals consider only construction costs, which leads to an incomplete vision of the water drive. This is a crucial error, since in 1994 Kim and Mays already reported that “for some utilities 90% of the total budget is for energy required for pumping” [3]. From 1980s onwards, some designers already integrate both the construction and the operating cost into the design procedure. Gessler and Walski [4] proposed a construction costs formulation including excellent details of digging costs and others. Alperovits and Shamir [5] detailed pumping costs calculated from an empirical approach that depends on the required power, providing practical results. Examples of recent designers that include both operational and construction costs are Ostfeld [6], who uses the software EPANET for the design; Pérez-Sánchez et al., Kang and Lansey, Jin et al. [7–9], and Samani and Mottaghi [10] carry out a “least cost assessment”, using hybrid programming. The remaining 30% of the known methodologies are multi objective proposals. The multi objective approaches are more widely followed in the past three decades and they consider the cost analysis and other aspects such as pressure deficit [11] or excess [12], greenhouse emissions [13], or water quality [14], among others. An excellent summary of the different approaches in multi objective methods is included in the work by Reed et al. [15].

In terms of the calculation process, evolutionary programming techniques like genetic algorithms have been widely used for water distribution optimization, for example by Marchi et al. [16], Kadu et al. [17], or Van Dijk et al. [18]. Nevertheless, most efforts in modern models deal with the computational processes. According to Goulter [19] most of the optimization models that are developed in research are not being used in the real practical design. The main reason of the limited use is not because the models do not work, as Walski et al. [20] proves, but mainly because of the challenges to interpret and use them in practical terms. In contrast, gradient search techniques are less common but they have convincingly shown that they can yield near optimal solutions for water networks [21]. They are also more intuitive than other design and optimization procedures.

On the other hand, due to climate change, population migrations, etc. [22], demand variations are becoming more extreme, increasing variability throughout the year [23,24]. This seasonal variation makes it even more complex for designers to properly conceive efficient water supply systems. For these reasons, designs that can manage changes in demand are desirable [25,26]. Babayan et al. [27] introduce this issue in their optimization using the standard deviation in the demand calculation. In studies like Granados et al. [28], safety coefficients are determined after a sensitivity analysis of the variables influencing the demand variation, especially for agricultural purposes. A different approach is used in Kapelan et al. [29], where the robustness is maximized generating random flow situations and calculating the probability to meet the required conditions at all nodes. Babayan et al. [30] compare the safety coefficient perspective to the approach of a random demand adjusting to a certain probability, and conclude that while both visions have similar results, the latter is more computationally expensive.

As suggestions for future works that will contribute to the crucial issue of the optimization of the design of water distribution systems for variable pumping flow rates, researches could also consider the assessment of the water sources and reclaimed water that can be used to balance water supply and demand [31].

The objective of this research is to develop a methodology that can overcome the limitations that a variable water demand regime brings to the design of a water supply system and that can easily be understood facilitating a practical approach. We propose two new design procedures that account for the variable demand, the Equivalent Flow Rate and the Equivalent Volume. The methodology proposed is based on a gradient based procedure, since it is very intuitive and not computationally demanding, therefore easily translated into practical terms. To illustrate the application, a case study is presented.

2. Materials and Methods

The total cost of a hydraulic impulsion depends mainly on the cost of construction of the pipe, the cost of construction of the pumping station, and the cost of the energy required for the pumps to work during the entire lifespan of the system. Other costs, such as maintenance and operation, may be neglected since they do not depend directly on pump or pipe sizing.

$$\text{Total Cost} = \text{Pipe Construction} + \text{Pumping Station Construction} + \text{Energy}$$

The Pipe Construction Cost increases as the pipe diameter widens, since the price of the tubes increases, and so too does the excavation and installation cost due to the difficulties that arise through transportation and assembly.

The construction cost of the pumping station can be considered relatively independent of that of the chosen pipe, since the different pump models usually have a similar cost and the group and stage configurations depend on the variability of the demand and not on the pipe features.

The cost of energy depends mainly on the volume of water to be elevated and also on the head and the pump efficiency. The head depends on the diameter of the pipe, since the larger the pipe diameter, the less head loss. In this way, using a larger diameter reduces the pumping cost. The efficiency of the pump is a variable whose relationship with the other factors had not been studied in depth until recently. As a general rule, it was assumed that the larger the pump is, the better performance it had. Nevertheless, Martin-Candilejo et al. [32] have addressed this issue in depth concluding that there is a direct relationship between the flow rate and the pump efficiency: The pump efficiency is better for greater discharge flow rates, reaching an asymptote for 90%.

Figure 1 is a scheme of the cost distribution depending on the pipe diameter. Since the pipes diameters do not form a continuous series, but rather are discrete values in the market, the traditional way of solving the problem has consisted various alternatives of diameters and pumps that meet the technical requirements and then each alternative is evaluated to select the one with the lowest cost.

Figure 1. Simplified scheme of the costs of the water drive depending on the diameter $\emptyset$ of the pipeline: As the diameter increases the construction costs rise but the head losses decrease, implying a reduction in the energy cost.

The procedure proposed in this work is based on the following principle: The head loss can be reduced by increasing the pipe diameter. This means savings in the energy cost but also an increase in the cost of construction. This can be studied from a unitary perspective: Comparing the cost of reducing one meter of head loss by increasing the pipe diameter with the savings in energy expenditure by not having to elevate water that one extra meter. This is the change Gradient concept, that was developed by Granados [33,34] as part of his pipe network optimization method, the Granados' System. The Change Gradient only makes sense when the

flow rate is constant. Therefore, for more realistic situations, this paper has developed the new concepts of the Equivalent Flow Rate and the Equivalent Volume for two different procedures for the design of a hydraulic drive.

2.1. Change Gradient Concept

The Change Gradient [33] is defined as the cost of reducing one meter of head loss by increasing the pipe diameter from ϕ_i to the next bigger one $\phi_j = \phi_{i+1}$:

$$GC_{\phi_i \rightarrow \phi_j}^q = \frac{P_j - P_i}{\Delta h_i^q - \Delta h_j^q}, \quad (1)$$

P_i , P_j are prices of pipes of length L and diameters ϕ_i and ϕ_j , respectively and Δh_i^q , Δh_j^q are the head losses of a pipe of length L and diameters ϕ_i and ϕ_j for a given q flow rate. This is represented in Figure 2.

Figure 2. Change Gradient concept. Increasing the pipe's diameter means a reduction in the head loss along the pipeline, but it has the extra cost of the wider and more expensive tube. The Change Gradient is the cost of reducing the head loss by 1 meter.

If, by instance, the head loss is calculated using Manning's formulation [34], the length of the pipe is canceled out from the Change Gradient's expression, and it stands as:

$$GC_{\phi_i \rightarrow \phi_j}^q = \frac{p_j L - p_i L}{\frac{L n_i^2 2^{20/3} q^2}{\pi^2 \phi_i^{16/3}} - \frac{L n_j^2 2^{20/3} q^2}{\pi^2 \phi_j^{16/3}}} = \frac{\pi^2}{n^2 2^{20/3}} \frac{(p_j - p_i)}{\left(\frac{n_i}{\phi_i^{16/3}} - \frac{n_j}{\phi_j^{16/3}} \right)} \frac{1}{q^2} = K_{GC} \frac{1}{q^2}, \quad (2)$$

being K_{GC} :

$$K_{GC} = \frac{\pi^2}{n^2 2^{20/3}} \frac{(p_j - p_i)}{\left(\frac{1}{\phi_i^{16/3}} - \frac{1}{\phi_j^{16/3}} \right)},$$

where L is the pipe length, p_i and p_j are the prices of the pipes of a diameter ϕ_i and ϕ_j , respectively; n_i and n_j are the Manning coefficients for the roughness of pipes i and j , respectively; and lastly q is the flow rate through the pipe.

Regarding the available diameters, each pipe manufacturer only offers a finite number of commercial diameters for each type of pipe. This means that the designer must adjust to the series of diameters offered for that pipe type.

Sometimes the pipes are conformed by two or more twin pipes, usually connected in parallel, that start from the same point and reach the same destination, as Figure 3 represents. In these cases, the flow rate through the drive q is distributed between these pipes, passing through each of them q/nt , where nt is the number of equal pipes that make up the drive. The price of

these pipes is nt times the price of an independent pipe. The Change Gradient corresponding this type of situations follows the following expression:

$$GC_{\phi_i \rightarrow \phi_j}^{q/nt} = \frac{\pi^2}{n^2 2^{20/3}} \frac{nt (p_j - p_i)}{\left(\frac{1}{\phi_i^{16/3}} - \frac{1}{\phi_j^{16/3}} \right)} \frac{1}{(q/nt)^2} = nt^3 GC_{\phi_i \rightarrow \phi_j}^{q/1t} \quad (3)$$

Figure 3. Parallel pipe configuration of the hydraulic drive.

2.2. Energy Cost

During the operation of a pumping station it is common that different discharges are pumped to meet the needs of the demand. The energy consumed in each of these situations is different, depending not only on the flow pumped, but also on the height pumped, the electromechanical efficiency, the duration of this pumping situation and the unit price of the energy.

If a simple case is analyzed, it can be assumed that there are n pumping situations that differ by the flow rate pumped ($q_1, q_2, q_i \dots q_n$). In this case, the annual cost for the energy consumed c_E can be obtained from the following expression [34] (the first and second equality are in International System units, whilst the third one depends on the following chosen units):

$$c_E = \sum E_i p_i = \sum_1^n \gamma q_i h_i \frac{1}{\mu_{B_i}} \frac{1}{\mu_{M_i}} t_i p_i = \sum_1^n 9.81 \frac{V_i}{3600} h_i \frac{1}{\mu_{B_i}} \frac{1}{\mu_{M_i}} p_i \quad (4)$$

where c_E is the annual cost of energy consumed by pumping (€); E is the annual energy consumed in the situation i (kWh); p_i is the unit price of energy in the situation i (€/kWh); γ is the specific weight of the pumped liquid (for water 9.810 N/m^3); q_i is the discharge pumped in the situation i (m^3/s); h_i is the height pumped in the situation i (m); μ_{B_i} and μ_{M_i} are the pump and engine efficiency in the situation i , respectively; t_i is the annual duration of situation i (hours); and lastly V_i is the annual volume (m^3) pumped during situation i .

This c_E cost is repeated every year during the lifespan of the pipe. To obtain the capitalized cost, the annual costs must be integrated at present value into a single global energy cost. The total cost of the energy spent during the useful life of the pipes capitalized at the beginning would therefore be C_E :

$$C_E = f_A c_E.$$

f_A is called discount factor and it depends mainly on the discount rate i , the lifespan of the pipe n_u and the construction period duration n_c . A generalist definition of f_A [34] would be:

$$f_A = \frac{(1+i)^{n_u} - 1}{(1+i)^{n_u} i} \times \frac{1}{(1+i)^{n_c}}.$$

2.3. Optimization for Constant Flow Rate

This situation is given when the pumping station works with a constant flow rate. This means the operating point of the pump does not change. In this way, all the other variables that affect the energy cost (q_i , h_i , $\otimes_B$, $\otimes_M$ and p_i) are constant. In this case, the flow rate q_i can be calculated, as well as the Change Gradients $GC_{\otimes_i \rightarrow \otimes_{i+1}}^q$ associated to the increase of diameter $\otimes_i$ to the next bigger one $\otimes_j = \otimes_{i+1}$. Likewise, as much as the cost of pumping energy is concerned, since the pump always works at the same operating point, the pump efficiency $\otimes_{Bi}$, the engine efficiency $\otimes_{Mi}$ and the pumping height h_i remain constant (they can be simplified to just h , $\otimes_B$, $\otimes_M$), and the total cost of energy C_E can be obtained with the following expression:

$$C_E = f_A C_E = f_A \sum_1^n 9.81 \frac{V_i}{3600} h_i \frac{1}{\mu_{Bi}} \frac{1}{\mu_{Mi}} p_i = f_A 9.81 \frac{\sum_1^n V_i}{3600} h \frac{1}{\mu_B} \frac{1}{\mu_M} p.$$

From the previous expression, making $h = 1\text{m}$ and $\sum_1^n V_i = V$ (which is the total annual volume pumped), the cost of the energy required for each meter of elevation C_{E1} can be calculated as:

$$C_{E1} = \frac{C_E}{h} = f_A 9.81 \frac{V}{3600} \frac{1}{\mu_B} \frac{1}{\mu_M} p. \quad (5)$$

With the exception of the pump performance $\otimes_B$, the other variables in the previous equation are actually data: V is the total volume to be pumped, p is the unit price of the energy that has been hired, and f_A is calculated from the discount rate i , the service life n_u of the water pipeline and the construction period n_c . Regarding the engine efficiency $\otimes_M$, although it varies theoretically depending on the engine model chosen and the operating point of the pump, the variations in engine performance are so small that it can be considered constant across different models and manufacturers [32].

Therefore, the above equation can be simplified by grouping all the data and parameters that have fixed values in the coefficient KC_{E1} , which can be considered constant for each case analyzed:

$$C_{E1} = KC_{E1} \frac{1}{\mu_B}, \quad (6)$$

$$KC_{E1} = f_A 9.81 \frac{V}{3600} \frac{1}{\mu_M} p. \quad (7)$$

In the previous expression, it is shown that the only variable that affects is the pump efficiency, μ_B , which will depend on the pump model chosen and the operating point (and it is yet unknown).

For this simple case of constant pumping flow, a design procedure for selecting the pipe diameter $\otimes$ is established through the following argument:

- If $GC_{\otimes_i \rightarrow \otimes_{i+1}}^q < C_{E1} \Rightarrow$ Diameter $\otimes_{i+1}$ is preferable to $\otimes_i$ since the cost of reducing 1 m the head loss by passing from $\otimes_{i+1}$ to $\otimes_i$ is cheaper than the cost of pumping that additional meter.
- If $GC_{\otimes_i \rightarrow \otimes_{i+1}}^q > C_{E1} \Rightarrow$ Diameter $\otimes_i$ is preferable to $\otimes_{i+1}$ since the cost of reducing 1 m the head loss by passing from $\otimes_{i+1}$ to $\otimes_i$ is more expensive than the cost of pumping that additional meter.

If this comparison is started on the diameter ϕ_1 , which is the smallest in the series of diameters available for that pipe model and manufacturer (that meets the maximum velocity condition along the network), the process leads to the optimum diameter for the water drive.

It has already been indicated that the value of C_{E1} depends on the pump model chosen, and more specifically, on its efficiency μ_B at the point of operation. Therefore, theoretically, the pump should first be chosen so that the pipe diameter can be selected. The μ_B efficiency at the operating point can only be obtained when the operating point is known, which depends on the diameter of the pipe. Therefore, the pipe diameter should first be known to obtain the operating point of the pump and thus its efficiency.

Therefore, starting by either selecting the pump, or the pipe diameter, the process involves iterating to find the optimal pipe diameter for each specific pump model. If the pump model changes, even slightly, the operating point would change and so would the pipe diameter.

In order to break this vicious circle, here we propose a more direct calculation process. It consists of solving the pump efficiency that each diameter requires to be competitive. As it is shown in the following equation, diameter ϕ_i will be the optimum (this means it should not be substituted by to the next diameter ϕ_{i+1} of the series) when:

$$GC_{\phi_i \rightarrow \phi_{i+1}}^q > C_{E1} \Rightarrow GC_{\phi_i \rightarrow \phi_{i+1}}^q > KC_{E1} \frac{1}{\mu_B} \Rightarrow \mu_B > \frac{KC_{E1}}{GC_{\phi_i \rightarrow \phi_{i+1}}^q} \Rightarrow \text{Keep } \phi_i, \quad (8)$$

otherwise:

$$GC_{\phi_i \rightarrow \phi_{i+1}}^q < C_{E1} \Rightarrow GC_{\phi_i \rightarrow \phi_{i+1}}^q < KC_{E1} \frac{1}{\mu_B} \Rightarrow \mu_B < \frac{KC_{E1}}{GC_{\phi_i \rightarrow \phi_{i+1}}^q} \Rightarrow \text{Move to } \phi_{i+1}. \quad (9)$$

This means that, whenever there is a pump on the market whose efficiency can be greater than the calculated μ_B , the optimum diameter will be ϕ_i . In the event that no commercial pump can reach that performance because it is very high, it will be necessary to move to the next diameter ϕ_{i+1} .

Therefore, to apply this method it is necessary to know the maximum performance that pumps can reach. Martin-Candilejo et al. [32] studied the optimum pump efficiency of 226 commercial pumps. After their assessment, they obtained an empiric relationship between the optimum μ_B and the flow rate, q . They presented Equation (10) to calculate the estimated optimum pump efficiency depending on the discharge flow. In their work, they also expose Equation (11) to determine the maximum expected value of the optimum μ_B .

$$\mu_B^{\text{Average}} = 0.1286 \ln (2.047 \ln q - 1.7951) + 0.5471; r^2 > 98\%, \quad (10)$$

$$\mu_B^{\text{Maximum}} = 0.0576 \ln (2.047 \ln q - 1.7951) + 0.741; r^2 > 90\%, \quad (11)$$

being q the circulating flow rate in liters per second (L/s).

Of course, only diameters that meet the minimum limitations required will be selected. These limitations might be maximum velocity, pressure [35,36], among others.

2.4. Optimization for Variable Flow Rate

Pumping stations that are designed to work with variable flow rates are more frequent in civil engineering applications, since they allow a better adjustment of the flow rates to those demanded at any time, avoiding in this way the situation of pumping a flow greater than the one required, and therefore, reducing the height losses with the consequent saving of energy. For this more realistic situation, the method proposed previously raises two problems:

- The Change Gradient can only be calculated at a constant flow rate.
- The energy cost C_{E1} is calculated for a constant pump performance and head.

To solve these two issues, this paper proposes two new methods called the Equivalent Flow Rate and the Equivalent Volume methods explained below.

2.4.1. Concept and Calculation of the Equivalent Flow Rate

The equivalent flow rate q_{Eq} can be defined as a theoretical flow rate for which, if all the volume V required in a year was pumped at this discharge, the cost of the pumping energy would be the same as the cost of pumping at a variable flow rate regime.

For the following reasoning, it is convenient to use a theoretical example. Assuming that the required annual volume V is pumped through a specific pipe and following a variable flow rate distribution, the annual energy cost c_E , applying the Equivalent Flow Rate definition, would result in:

$$c_E = \sum_{k=1}^n 9.81 \frac{V_k}{3600} h_k \frac{1}{\mu_{Bk}} \frac{1}{\mu_{Mk}} p_k = 9.81 \frac{V}{3600} h_{Eq} \frac{1}{\mu_{BEq}} \frac{1}{\mu_{MEq}} p_{Eq} \quad (12)$$

being V the annual volume of water pumped; h_{Eq} the pumping head for the theoretical operating point corresponding to q_{Eq} ; μ_{BEq} and μ_{MEq} the pump and engine efficiency at the theoretical operating point corresponding to q_{Eq} ; n the number of periods of different flow rate, and p_{Eq} the theoretical unit price of the energy with which q_{Eq} would be pumped. Figure 4 represents this idea the concept of the equivalent flow rate.

Figure 4. Equivalent Flow Rate definition.

For this analysis, the cost of pumping energy should be separated into two parts. On the one hand, the cost corresponding strictly to the geometric height, and on the other hand, the cost corresponding to the head losses. In this way, knowing that $h_k = h_G + \Delta h_k$, the previous equation would be:

$$c_E = \sum_{k=1}^n 9.81 \frac{V_k}{3600} (h_G + \Delta h_k) \frac{1}{\mu_{Bk}} \frac{1}{\mu_{Mk}} p_k = 9.81 \frac{V}{3600} (h_G + \Delta h_{Eq}) \frac{1}{\mu_{BEq}} \frac{1}{\mu_{MEq}} p_{Eq} \quad (13)$$

where h_G is the geometric height and Δh_{Eq} is the head losses when q_{Eq} is circulating.

Some simplifications can be made in the previous equation. To begin with, the motor efficiency and the unit price of energy can be assumed as constant. These assumptions are quite close to reality, since, as it has been already indicated, motor efficiencies vary very little in different operating regimes, and it is common to hire a flat rate of energy. With the above simplifications, the expression would be:

$$V h_G + V \Delta h_{Eq} = \sum_{k=1}^n \frac{\mu_{BEq}}{\mu_{Bk}} V_k h_G + \sum_{k=1}^n \frac{\mu_{BEq}}{\mu_{Bk}} V_k \Delta h_k$$

Now, a new simplification must be made, and this one is more debatable. It consists of assuming a constant pump efficiency. This implies that all η_{Bk} are equal, including μ_{BEq} . This is a purely operational assumption and does not correspond to reality, since the pump's performance varies significantly for the different operating points. This simplification is made in order to prevent from future iterations to obtain the Equivalent Flow Rate, and it is later shown to be unnecessary.

The general expression would be:

$$V h_G + V \Delta h_{Eq} = \sum_1^n V_k h_G + \sum_1^n V_k \Delta h_k \Rightarrow V \Delta h_{Eq} = \sum_1^n V_k \Delta h_k.$$

On the other hand, the various head losses associated to different flow rates can be expressed as below using Manning's expression:

$$\Delta h_k = \frac{L n^2 2^{20/3} q_k^2}{\pi^2 \phi^{16/3}} = \alpha q_k^2. \quad (13)$$

Applying this to the previous equation, the new general expression is:

$$V \alpha q_{Eq}^2 = \sum_1^n V_k \alpha q_k^2,$$

and it can finally be simplified to obtain the Equivalent Flow Rate:

$$q_{Eq} = \sqrt{\frac{\sum_1^n V_k q_k^2}{V}}, \quad (14)$$

where q_{Eq} is the Equivalent Flow Rate. It is a constant flow that implies the same energy cost as that of a variable flow regime; q_k is the flow pumped in the operating situation k ; $V_k = q_k \times t_k$, is the annual volume in operating situation k ; t_k is the annual time spent in pumping q_k ; and at last, V is the annual volume pumped, calculated as $V = \sum_1^n V_k$.

The previous expression shows that the Equivalent Flow Rate is an average flow rate of the variable regime. From this point of view, it is an intermediate discharge that should correspond to an operating point close to the optimum of the pump, in a way that the real variable flow values are located around the sides of this optimum.

It should be remembered that the previous equivalent flow rate has been obtained by making three assumptions: that the engine efficiency, that the unit price of energy, and that the pump efficiency are the same for any operating point. The first two are logical and respond to the reality of most cases. The third one is, however, conceptually incorrect and it would theoretically require a second approximation of the Equivalent Flow Rate once the pipe diameter has been obtained and the pump selected. However, our simulations let us affirm that this first approximation of the equivalent flow rate is already sufficient, since the pump performances, although different, compensate between the different operating points if the pump is properly selected.

2.4.2. Optimization Procedure Using the Equivalent Flow Rate Method

The Equivalent Flow Rate has been developed to calculate the Change Gradients when the discharge regime is variable. As previously explained, each Change Gradient is associated with a specific flow rate. Therefore, when the flow regime is variable, it is necessary to use the Equivalent Flow Rate to calculate them. In this way, the expression of the Change Gradient will change to:

$$GC_{\phi_i \rightarrow \phi_j}^{q_{Eq}} = \frac{P_j - P_i}{\Delta h_i^{q_{Eq}} - \Delta h_j^{q_{Eq}}}.$$

Using the concept of the Equivalent Flow Rate in the calculation of the cost of energy per meter lifted, the previous equation turns into:

$$C_{E1} = \frac{C_E}{h_{Eq}} = f_A 9.81 \frac{V}{3600} \frac{1}{\mu_{BEq}} \frac{1}{\mu_{MEq}} p_{Eq} = KC_{E1}^{q_{Eq}} \frac{1}{\mu_{BEq}}.$$

It has already been discussed that the engine efficiency and the unit price of energy can be assumed equal for any operating point. For this reason, coefficient $KC_{E1}^{q_{Eq}}$ does not really depend on the value of the Equivalent Flow Rate, so it will remain as KC_{E1} .

Therefore, the Equivalent Flow Rate allows the calculation of the Change Gradients and the Energy Cost per meter pumped for a variable discharge regime. The comparison between the two of these two terms allows to obtain the optimum diameter of the pipeline, following the same reasoning that was previously described: Diameter ϕ_i will be optimum (e.g., it should not be passed onto the next diameter of the series ϕ_{i+1}) when:

$$GC_{\phi_i \rightarrow \phi_{i+1}}^{q_{Eq}} > C_{E1} \Rightarrow GC_{\phi_i \rightarrow \phi_{i+1}}^{q_{Eq}} > KC_{E1} \frac{1}{\mu_{Bq}} \Rightarrow \mu_{Bq} > \frac{KC_{E1}}{GC_{\phi_i \rightarrow \phi_{i+1}}^{q_{Eq}}} \Rightarrow \text{Select } \phi_i.$$

As a summary, the method would be:

1. Equivalent Flow Rate is calculated:

$$q_{Eq} = \sqrt{\frac{\sum_1^n V_k q_k^2}{V}}.$$

2. With q_{Eq} , the Change Gradients are calculated for each increase in diameter (starting with the smaller in a commercial list and passing onto the very next one).

$$GC_{\phi_i \rightarrow \phi_{i+1}}^{q_{Eq}} = \frac{\pi^2}{n^2 2^{20/3}} \frac{(p_j - p_i)}{\left(\frac{1}{\phi_i^{16/3}} - \frac{1}{\phi_{i+1}^{16/3}} \right)} \frac{1}{q_{Eq}^2}.$$

3. Parameter KC_{E1} is calculated:

$$KC_{E1} = f_A \frac{9.81}{3600} \frac{V}{\mu_M} p.$$

4. The required pump efficiency corresponding to each diameter change is calculated:

$$\mu_B = \frac{KC_{E1}}{GC_{\phi_i \rightarrow \phi_{i+1}}^{q_{Eq}}}.$$

5. When the pump efficiency reaches a value that can be easily found in the pump market, diameter ϕ_i is selected.
6. In case the pump efficiency seems too high to reach, it is preferable to select a bigger diameter ϕ_{i+1} . The initial investment will be greater, but the cost of pumping energy will be lower. This reduces the risk in the event of rises in energy price, different from the predictions made during design phase. As a reference for the expected value of the pump efficiency use Equation (10).

To sum up, the optimization procedure proposed in this section is based on the two following premises, which have been demonstrated during the development of the work itself:

- The optimum diameter of the water drive depends only on the pump performance.
- The cost of the full water drive (pipes and pumps) depends only on the pipe diameter.

Therefore, the optimum design of a water drive depends on one only variable, and that is the flow rate. If, as usual, the flow rate is variable, the Equivalent Flow Rate can be used, whose formulation is an original innovation of this work.

2.4.3. Concept and Calculation of the Equivalent Volume

If a constant flow rate q_m is chosen to design the pipe line (preferable $q_m = 1 \text{ m}^3/\text{s}$ to make calculations easier), $V_{Eq}^{q_m}$ is the total Equivalent Volume of water that needs to be pumped at that flow rate q_m to make the final cost of pumping equal to what it would be pumping at a variable regime of flow rates. For the calculation of the Equivalent Volume $V_{Eq}^{q_m}$, a similar reasoning to the one followed for the Change Gradient will be applied: The final cost will always be referred to the cost of elevating the water at 1m height C_{E1} . Applying this approach in Equation (5), C_{E1} can be expressed in terms of $V_{Eq}^{q_m}$ as below:

$$C_{E1} = f_A 9.81 \frac{V_{Eq}^{qm}}{3600} \frac{1}{\mu_B} \frac{1}{\mu_M} p = K_{Cv} \frac{V_{Eq}^{qm}}{\mu_B}, \quad (15)$$

$$K_{Cv} = f_A 9.81 \frac{1}{3600} \frac{1}{\mu_M} p.$$

Therefore, the concept of the Equivalent Volume is expressed as follows.

$$C_{E1} = \sum C_{E1}^{qk} = C_{E1}^{qm} = K_{Cv} \frac{V_{Eq}^{qm}}{\mu_B}. \quad (16)$$

Apart from that, in the situation that a flow rate q_i is circulating, an increase of the pipe diameter from ϕ_i to ϕ_{i+1} would mean a reduction of the head loss $\Delta\Delta h_{\phi_i \rightarrow \phi_{i+1}}^{qk}$ that, using Manning's expression, only depends on the flow discharge q_i . To simplify the terminology $\Delta\Delta h_{\phi_i \rightarrow \phi_{i+1}}^{qk}$ will be referred as $\Delta\Delta h^{qk}$.

$$\Delta\Delta h_{\phi_i \rightarrow \phi_{i+1}}^{qk} = \Delta\Delta h^{qk} = \Delta h_{\phi_i}^{qk} - \Delta h_{\phi_{i+1}}^{qk} = \beta \times q_k^2,$$

being β a constant deduced from the invariable terms in Manning's formula (see Equation (13)).

This reduction of the head loss translates in a savings in the energy cost $C_{\Delta E}^{qk}$. To express $C_{\Delta E}^{qk}$, the same simplifications that were used for the Equivalent Flow Rate will be made: the engine efficiency, that the unit price of energy, and that the pump efficiency are the same for any operating point. Once again, the first two assumptions are close to reality, but the third one is not; however further simulations have shown that the pump performances, although different, compensate between the different operating points. This being said, the savings in energy costs can be expressed as it follows.

$$C_{\Delta E}^{qk} = f_A 9.81 \frac{V_k}{3600} \Delta\Delta h^{qk} \frac{1}{\mu_{Bk}} \frac{1}{\mu_{Mk}} p_k = K_{Cv} V_k \Delta\Delta h^{qk} \frac{1}{\mu_B}. \quad (17)$$

Additionally, any variance of the head loss associated to a flow rate can be expressed in terms of a different discharge as:

$$\Delta\Delta h^{qk} = \beta \times q_k^2 \rightarrow \Delta\Delta h^{qk} = \beta \frac{q_m^2}{q_m^2} q_k^2 = \Delta\Delta h^{qm} \frac{q_k^2}{q_m^2}.$$

Introducing the previous expression in Equation (16), $C_{\Delta E}^{qi}$ is then:

$$C_{\Delta E}^{qk} = K_{Cv} V_k \Delta\Delta h^{qm} \frac{q_k^2}{q_m^2} \frac{1}{\mu_B}.$$

However, as it was expressed in the Equivalent Volume definition, this needs to be analyzed from the perspective of pumping water at 1 m height so that, later on, the Change Gradient methodology can be followed. For this unitary point of view, C_{E1}^{qi} is obtained from $C_{\Delta E}^{qi}$ by:

$$C_{E1} = \frac{\sum C_{\Delta E}^{qk}}{\Delta\Delta h^{qm}} = K_{Cv} \frac{\sum V_k q_k^2}{q_m^2} \frac{1}{\mu_B}.$$

Introducing this conclusion in Equation (13), the Equivalent Volume formulation can be deduced:

$$K_{Cv} \frac{V_{Eq}^{qm}}{\mu_B} = K_{Cv} \frac{\sum V_k q_k^2}{q_m^2} \frac{1}{\mu_B}.$$

Therefore, the Equivalent Volume is:

$$V_{Eq}^{qm} = \frac{\sum V_k q_k^2}{q_m^2}.$$

This expression is simplified if the virtual constant discharge of design q_m takes the value of $q_m = 1 \text{ m}^3/\text{s}$. For this value, the Equivalent Volume is simplified as:

$$V_{Eq}^{qm=1 \text{ m}^3/\text{s}} = \sum V_k q_k^2. \quad (18)$$

2.4.4. Optimization Procedure using the Equivalent Volume Method

The Equivalent Volume has been deduced to calculate the Change Gradients when the discharge regime is variable. As a summary, the process is:

1. Choose any value for a virtual constant flow rate q_m . It is recommended that $q_m = 1 \text{ m}^3/\text{s}$. All of the following steps will be shown for this value.
2. For the chosen q_m calculate the Change Gradients series, correspondent to the change of one pipe diameter from a commercial catalogue to the immediate bigger one:

$$GC_{\phi_i \rightarrow \phi_{i+1}}^{q=1} = \frac{\pi^2}{n^2 2^{20/3}} \frac{(p_j - p_i)}{\left(\frac{1}{\phi_i^{16/3}} - \frac{1}{\phi_j^{16/3}} \right)}.$$

3. Calculate the Equivalent Volume for q_m :

$$V_{Eq}^{q_m} = \frac{\sum V_k q_k^2}{q_m^2} \rightarrow V_{Eq}^{q_m=1 \text{ m}^3/\text{s}} = \sum V_k q_k^2.$$

4. Calculate KC_{E1} for $V_{Eq}^{q_m}$:

$$KC_{E1} = \frac{9.81 f_A p}{3600 \mu_M} V_{Eq}^{q_m}.$$

5. Calculate the required pump efficiency needed to each diameter change:

$$\mu_B = \frac{KC_{E1}}{GC_{\phi_i \rightarrow \phi_{i+1}}^{q_{Eq}}}.$$

6. At the point, the same reasoning as for the constant discharge situation is followed: When the pump efficiency reaches a value that could easily be found in the market (e.g., between 80–85%), select diameter ϕ_i . For those cases of uncertainty, it is preferable to select a bigger diameter ϕ_{i+1} . The initial investment will be bigger but the risk of additional cost in case of a higher energy price will be diminished.

3. Results and Discussion

Case study: Parallel pipes using the Equivalent Flow Rate

As a simple example of the applications of the Equivalent Flow Rate concept, a theoretical case of a hydraulic drive has been studied. The drive consists of two parallel pipes of 500 m long. The water drive serves agricultural purposes, and the total area to supply is 3000 ha. The system has no regulation at the end of the drive, as Figure 5 illustrates, and therefore, the flow rate will vary depending on the demand of the month, therefore, the scheme is similar to Figure 3 (without regulation tank at the end of the circuit). As it is expected, this demand is higher for the warmer periods (reaching its peak in July), and will stop during winter, conforming in this way, the hydrological year. The demand distribution is shown in Table 1 and the average monthly flow rate q_k is calculated below.

This illustrates a classical case very common in the professional practice, which is the elevation of water to a higher deposit. In practice, the designing would firstly require the pipeline price collection among the local manufacturers as well as a proper demand study, so that the design flow rate can be obtained. With this data, the procedure will follow the steps enumerated in Section 2.4.2.

Figure 5. Case study scheme for application of the Equivalent Flow Rate concept.

Table 1. Water demand in the study case.

Real Water Demand								
Time: t_k	Month	April	May	June	July	August	September	Total
	Days	30	31	30	31	31	30	183
Water	m^3/ha	500	1000	1000	1500	1000	500	5500
Provision	$m^3/month$	1,500,000	3,000,000	3,000,000	4,500,000	3,000,000	1,500,000	16,500,000
q_k	m^3/s	0.58	1.12	1.16	1.68	1.12	0.58	1.044

The new approach derived in this study and presented in the Methodology section is applied in the case study, where the circulating flow rate is variable, and this prevents from direct calculation (see data in Table 1). Therefore, to begin with, applying Equation (14), a constant flow rate distribution that is equivalent in energy costs to the real one is obtained: The Equivalent Flow Rate is calculated, as shown in Table 2.

Table 2. Equivalent flow rate calculation from Equation (13).

Equivalent Flow Rate						
$V_k \times q_k^2$						q_{eq}
April	May	June	July	August	September	(m^3/s)
502,347	3,763,682	4,018,776	12,702,426	3,763,682	502,347	1.237

Once the homogeneous demand distribution has been obtained, thanks to the Equivalent Flow Rate, Granados System can be applied. As it was previously explained, the methodology consists of comparing the cost of investing in buying a bigger pipe or pumping greater head losses, and choosing whatever is cheaper. This analysis is made through the Change Gradient concept, and it requires a commercial series of diameters and its prices. In Table 3, it was included a list of diameters and its correspondent prices as a representation of a commercial catalog. These prices have been estimated based on average values for the selected material, which in this case stainless steel has been preferred. For Manning Coefficient, the selection of the friction factor a value of 0.0085 has been assumed for stainless steel pipes in good state. In this way, all Change

Gradients are calculated following Equation (3). Table 3 shows the resulting $GC_{\phi_i \rightarrow \phi_{i+1}}^{qEq}$ for each diameter onto the next one.

Table 3. Commercial diameter series accompanied by their correspondent price. Change Gradient calculation and optimization process by calculating the required efficiency of the pump.

Pipe Catalog		Optimization		
Stainless Steel		Change Gradient		Efficiency
Diameter	Price	One Pipe	Parallel Pipes	μ_B
mm	€/m	€/m	€/m	%
300	57.5	11.6	46.6	179937%
400	63.9	220.8	883.3	9487%
500	87.1	998.0	3992.1	2099%
600	115.6	3185.7	12,742.9	658%
700	146.6	8117.1	32,468.6	258%
800	178.1	25,318.6	101,274.6	83%
900	222.3	50,705.5	202,822.1	41%
1000	265.8	100,827.2	403,308.8	21%
1100	311.6	193,612.7	774,450.8	11%
1200	360.8	313,524.1	1,254,096.4	7%
1300	407.7	517,682.7	2,070,730.7	4%
1400	455.1			

These $GC_{\phi_i \rightarrow \phi_{i+1}}^{qEq}$ need to be compared with the cost of pumping the full annual volume at 1 meter height. As it was previously explained, because this annual cost depends on the pump efficiency μ_B , the analysis turns into seeing what μ_B needs to be for the pumping option to be cheaper than building a greater pipe. For this analysis KCE1 is calculated, following Equation (7), as it is shown in Table 4. In this case study, it has been assumed that the construction period would last for two years, the useful life of the installation will be 25 years, and that a flat rate is hired, being 0.12 €/kWh the accorded price for the energy. The engine efficiency is taken as 0.93 and, in order to update the cost along the useful life, 4% is assumed as the discount rate. All of these values are estimated as usual values in the field.

Table 4. Energy cost constant factors and calculation of the invariable part of C_{E1} .

Energy Cost						
Energy Price	Construction Years	Useful Life	Engine Efficiency	Discount Rate	Discount Factor	KC_{E1} Constant
p_e	n_c	n_u	μ_M	i	f_a	KC_{E1}
(€/kWh)	(years)	(years)	(%)	(%)		(€/m)
0.12	2	25	0.93	0.04	14.44	83,796

Once all $GC_{\phi_i \rightarrow \phi_{i+1}}^{qEq}$ and KC_{E1} have been calculated, the following step is to obtain the efficiency required for the pumps so that the energy cost is smaller than the construction cost. As it can be observed in Table 3 and in Figure 6, a diameter of 800 mm requires a pump efficiency of 83% which is a value that can easily be found in the market. Per contrary, a diameter of 700 mm is too small because it would require a pump efficiency of 258%, which is completely irrational. On the other hand, a pipe of 900 mm only requires 41% efficiency, so it is already too wide and it

is more convenient to pump the head losses produced by an 800 mm pipe, than to invest in a wider pipe (as a 900 mm, per instance). Therefore, the optimum pump diameter of the water drive would be 800 mm wide

Figure 6. Pump efficiency required for each diameter. For the construction cost to be competitive with the energy cost, each pipeline designed with diameter $\varnothing$ requires a pump with a minimum efficiency of μ_B . The smaller the pipe, the better (since the construction costs will be smaller), but the required μ_B should be available in the market. Therefore, diameter $\varnothing$ 800 mm is selected since it is the smallest of the commercial catalog meeting realistic performance conditions.

As it can be seen in Figure 7, the Change Gradient shows that, starting the design from small pipes, it is very cheap and convenient to move to bigger diameters, but the construction cost will rise up as the pipe gets wider. At some point the cost of increasing the pipe diameter to reduce the head loss will be too high, and it will be better to pump the water, as long as the pump efficiency has a reasonable value.

Figure 7. Change Gradient and pipe diameter in the application case.

However, it is important to remind that the simplicity of the Granados' System requires a constant demand distribution, and it would not be possible to apply for a variable flow rate as the one of this study case, were it not for the Equivalent Flow Rate concept, which is a novelty of this research study. Combined with the Granados' System, the application of the Equivalent Flow

Rate (or the Equivalent Flow Rate) give practice engineers a simple design alternative to obtain the main initial figures to work with.

Figure 8 (a) and (b) show the variable real demand distribution compared with the constant equivalent one, and in it can be observed that both distributions mean the same annual cost for the energy required in the water drive. As Figure 9 shows, the minimum total cost of the installation is achieved with diameter 800 mm, and for that reason it is the one to be selected for the design of the water drive. The graph agrees with the result previously given by the method. In Figure 8 (c) and (d), the separated construction and energy costs are shown.

Figure 8. (a) Flow rate distribution in the application case of two parallel tubes and (b) annual accumulated cost of the energy required for the head losses. (c) Construction cost of the installation depending on the pipe diameter. (d) Accumulated energy cost throughout the entire lifespan of the installation depending on the pipe diameter.

Figure 9. Total accumulated cost of the water drive throughout the entire lifespan of the installation depending on the pipe diameter. Dimeter Ø800 provides the smallest total cost for the construction and operation altogether, and therefore it is the one selected, as the method previously anticipated.

4. Conclusions

The study presents a practical approach that allows to calculate variable flow rates in a practical way since: (a) actual computational resources allow to work with static variables although the mathematical approach of the Granados' System is dynamic; and (b) it is efficient since it only needs a few operations up to the optimum, making the proposed procedure extremely computationally straight forward. The approach avoids the major computational inconvenience of dynamic programming that may limit the use in practical designs.

Our novel concepts applied in the proposed approach of the Equivalent Flow Rate and the Equivalent Volume offer engineers a solution to the main constrain of the cost gradient techniques, which rely on constant design flow rates. The uncertainties regarding the future water demand and the evolution of the energy price are a major concern in nowadays water supply designs and there is still effort to make to accurately determine what the demand curve will be in the fore coming years; however, these two concepts allow the designer to include various different flow rates along the time, contributing to the mitigation of this issue regarding the design. Future research should focus on probabilistic methods to determine the demand pattern for the fore coming years.

The present procedure is similar in concept to the ratio used by [19] in their proposal for optimization of the design of water supply system. However, their system also depends on a constant flow rate, and the novel concepts of this paper of the Equivalent Flow Rate and Equivalent Volume is also applicable for their algorithm.

The applications of the present methodology are adequate for branched networks, including those with a twin pipe configuration, penstock and hydroelectric power plants, and the approach is also extensible to channel design. Regardless of the fact this report does not present a solution for all networks, it covers a wide range of configurations. As Walski [37] states "chances that a single optimization approach will work for all types of problem is unlikely, and it is somewhat understandable that practicing engineers look skeptically at models that claim to optimize pipe selection".

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References

- Colglazier, W. Sustainable development agenda: 2030. *Science* **2015**, *349*, 1048–1050.
- Mala-Jetmarova, H.; Sultanova, N.; Savic, D. Lost in optimisation of water distribution systems? A literature review of system operation. *Environ. Model. Softw.* **2017**, *93*, 209–254.
- Kim, J.H.; Mays, L.W. Optimal Rehabilitation Model for Water-Distribution Systems. *J. Water Resour. Plan. Manag.* **1994**, *120*, 674–692.
- Gessler, J.; Walski, T.M. *Water Distribution System Optimization*; Army Engineer Waterways Experiment Station Vicksburg Ms Environmental Lab: Mississippi, USA, **1985**.
- Alperovits, E.; Shamir, U. Design of optimal water distribution systems. *Water Resour. Res.* **1977**, *13*, 885–900.
- Ostfeld, A. Optimal Design and Operation of Multiquality Networks under Unsteady Conditions. *J. Water Resour. Plan. Manag.* **2005**, *131*, 116–124.
- Pérez-Sánchez, M.; Sánchez-Romero, F.J.; Ramos, H.M.; López-Jiménez, P.A. Calibrating a flow model in an irrigation network: Case study in Alicante, Spain. *Span. J. Agric. Res.* **2017**, *15*, e1202.
- Kang, D.; Lansey, K., Revisiting optimal water-distribution system design: issues and a heuristic hierarchical approach. *J. Water Recour. Plan. Manag.* **2011**, *138*, 208–217.
- Jin, X.; Zhang, J.; Gao, J.-L.; Wu, W.-Y. Multi-objective optimization of water supply network rehabilitation with non-dominated sorting Genetic Algorithm-II. *J. Zhejiang Univ. A* **2008**, *9*, 391–400.
- Samani, H.M.V.; Mottaghi, A. Optimization of Water Distribution Networks Using Integer Linear Programming. *J. Hydraul. Eng.* **2006**, *132*, 501–509.
- Cisty, M.; Bajtek, Z.; Celar, L. A two-stage evolutionary optimization approach for an irrigation system design. *J. Hydroinformatics* **2016**, *19*, 115–122.
- Zheng, F.; Simpson, A.; Zecchin, A. Improving the efficiency of multi-objective evolutionary algorithms through decomposition: An application to water distribution network design. *Environ. Model. Softw.* **2015**, *69*, 240–252.
- Stokes, C.S.; Maier, H.R.; Simpson, A.R., Effect of storage tank size on the minimization of water distribution system cost and greenhouse gas emissions while considering time-dependent emissions factors. *J. Water Resour. Plan. Manag.* **2015**, *142*, 04015052.
- Shokoohi, M.; Tabesh, M.; Nazif, S.; Dini, M., Water quality based multi-objective optimal design of water distribution systems. *Water Resour. Manag.* **2017**, *31*, 93–108.
- Reed, P.M.; Hadka, D.; Herman, J.D.; Kasprzyk, J.R.; Kollat, J.B., Evolutionary multiobjective optimization in water resources: The past, present, and future. *Adv. Water Resour.* **2013**, *51*, 438–456.
- Marchi, A.; Dandy, G.; Wilkins, A.; Rohrlach, H., Methodology for comparing evolutionary algorithms for optimization of water distribution systems. *J. Water Resour. Plan. Manag.* **2012**, *140*, 22–31.
- Kadu, M.S.; Gupta, R.; Bhave, P.R., Optimal design of water networks using a modified genetic algorithm with reduction in search space. *J. Water Resour. Plan. Manag.* **2008**, *134*, 147–160.
- Van Dijk, M.; van Vuuren, S.J.; Van Zyl, J.E., Optimising water distribution systems using a weighted penalty in a genetic algorithm. *Water SA* **2008**, *34*, 537–548.
- Goulter, I., Systems analysis in water-distribution network design: From theory to practice. *J. Water Resour. Plan. Manag.* **1992**, *118*, 238–248.
- Walski, T.M.; Brill Jr, E.D.; Gessler, J.; Goulter, I.C.; Jeppson, R.M.; Lansey, K.; Lee, H.-L.; Liebman, J.C.; Mays, L.; Morgan, D.R., Battle of the network models: Epilogue. *J. Water Resour. Plan. Manag.* **1987**, *113*, 191–203.
- Dziedzic, R.; Karney, B.W., Cost gradient-based assessment and design improvement technique for water distribution networks with varying loads. *J. Water Resour. Plan. Manag.* **2015**, *142*, 04015043.
- Romero Marrero, L.E.; Pérez-Sánchez, M.; López Jiménez, P.A., Water implications in Mediterranean irrigation networks: Problems and solutions. *Int. J. Energy Environ.* **2017**, *8*, 83–96.
- Spiliotis, M.; Iglesias, A.; Garrote, L. A Meta-multicriteria Approach to Estimate Drought Vulnerability Based on Fuzzy Pattern Recognition. In Proceedings of the Communications in Computer and

- Information Science, Xersonisos, May 2019; Springer Science and Business Media LLC: Crete, Greece, 2019; pp. 349–360.
24. Spiliotis, M.; Papadopoulos, B.K., A hybrid fuzzy probabilistic assessment of the extreme hydrological events. In *International Conference of Numerical Analysis and Applied Mathematics (AIP Conference Proceedings)* **2017**. AIP Publishing: Thessaloniki, Greece, 25 September 2017.
 25. Tsakiris, G.; Spiliotis, M. Uncertainty in the analysis of urban water supply and distribution systems. *J. Hydroinformatics* **2017**, *19*, 823–837.
 26. Spiliotis, M.; Tsakiris, G., Incorporating uncertainty in the design of water distribution systems. *Ewra* **2017**, *European Water*, 58: 449–456.
 27. Babayan, A.; Kapelan, Z.; Savić, D.; Walters, G. Least-Cost Design of Water Distribution Networks under Demand Uncertainty. *J. Water Resour. Plan. Manag.* **2005**, *131*, 375–382.
 28. Granados, A.; Martín-Carrasco, F.J.; De Jalón, S.G.; Iglesias, A. Adaptation of irrigation networks to climate change: Linking robust design and stakeholder contribution. *Span. J. Agric. Res.* **2015**, *13*, 32.
 29. Kapelan, Z.S.; Savić, D.A.; Walters, G.A. Multiobjective design of water distribution systems under uncertainty. *Water Resour. Res.* **2005**, *41*, doi: 10.1029/2004WR003787.
 30. Babayan, A.V.; Savić, D.A.; Walters, G.A.; Kapelan, Z.S. Robust Least-Cost Design of Water Distribution Networks Using Redundancy and Integration-Based Methodologies. *J. Water Resour. Plan. Manag.* **2007**, *133*, 67–77.
 31. Rak, J.R.; Pietrucha-Urbanik, K. An Approach to Determine Risk Indices for Drinking Water—Study Investigation. *Sustain.* **2019**, *11*, 3189.
 32. Martin-Candilejo, A.; Santillán, D.; Garrote, L. Pump Efficiency Analysis for Proper Energy Assessment in Optimization of Water Supply Systems. *Water* **2019**, *12*, 132.
 33. Granados, A., *Problemas de Obras Hidráulicas*, Colegio de Ingenieros de Caminos, Canales y Puertos, Madrid, ISBN 978-84-380-0090-8, **1995**.
 34. Carrasco, F. J. M.; Garrote, L. d., *Dimensionamiento y Optimización de Obras Hidráulicas*. 4th ed.; Editorial Garceta, Colegio de Ingenieros de Caminos, Canales y Puertos: Madrid, Spain' ISBN: 978-84-1545-254-6; **2013**.
 35. Vicente, D.; Garrote, L.; Sánchez, R.; Santillán, D., Pressure management in water distribution systems: Current status, proposals, and future trends. *J. Water Resour. Plan. Manag.* **2015**, *142*, 04015061.
 36. Pérez-Sánchez, M.; López-Jiménez, P.A.; Ramos, H.M. PATs Operating in Water Networks under Unsteady Flow Conditions: Control Valve Manoeuvre and Overspeed Effect. *Water* **2018**, *10*, 529.
 37. Walski, T.M. Optimization and Pipe-Sizing Decisions. *J. Water Resour. Plan. Manag.* **1995**, *121*, 340–343.

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