

# Multi-Objective Design of District Metered Areas in Water Distribution Networks



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*to Marcella and Raffaele*



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# Abstract

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The present thesis addresses the sectorization of Water Distribution Networks (WDNs), a technique based on the use of shut-off valves for the establishment of controlled sub-zones, usually called District Metered Areas (DMAs).

The DMA approach is widely recognized as one of the most successful and cost-effective methods for the optimization of the WDNs. Among the many benefits achievable through the sectorization, the most interesting effects are the increased system control and the contribution offered to the mitigation of the water losses.

The establishment of DMAs can be carried out under several perspectives and with different goals. Moreover, in real cases the achievement of the optimal design of DMAs may be very challenging because of the intrinsic complexity of the WDNs. Therefore, the development of a methodology able to provide support to decision-making is required.

In recent years, an increasing number of researches have addressed this problem, and different optimization approaches can be found in literature. However, the techniques developed so far still suffer from some limitations and drawbacks, which mainly consist in the lack of engineering principles among the design criteria and the dependence on the size of the problem.

In this thesis, a comprehensive methodology for the optimal sectorization of WDNs is presented and discussed. The proposed approach consists of a two-objectives optimization problem subjected to a number of constraints related to the topology of the WDN, financial issues and the network hydraulics.

One of the most innovative feature of this method is the particular emphasis placed on the minimization of the water leakages, that is explicitly included in the objectives. As highlighted in some studies, the establishment of DMAs is crucial for the development of optimization approaches based on pressure control (e.g. use of Pressure Reducing Valves, PRVs). However, sectorization itself can be considered as a pressure management technique. In fact, the closure of a number of network pipes may cause a significant increase of the head-losses throughout the WDN, with the consequent decrease of the pressures and of the water leakages.

A Multi-Objective Evolutionary Algorithm (MOEA) is combined with tools from graph theory for the solution of the problem. The procedure is completely implemented in Matlab environment, and it allows the automatic generation of feasible and optimal designs of DMAs just basing on the network model.

The validation of the methodology and the calibration of the parameters are performed through the application to a well-known example from literature. Then, a real test case is analyzed, namely the WDN of Pianura, a neighbourhood in the city of Naples, Italy. Low cost as compared to the budget and a good saving in leakage are obtained.

**Keywords:** District Metered Areas, sectorization of water networks, pressure management, water losses, multi-objective optimization.



# Table of contents

---

Chapter 1 – INTRODUCTION.....	1
1.1. Problem statement .....	3
1.2. Purpose of the work.....	4
1.3. Outline of the thesis .....	5
References .....	7
Chapter 2 – MANAGEMENT OF WATER SYSTEMS .....	9
2.1. Water losses in supply and distribution systems .....	14
2.2. Management of water networks in the Italian legislation .....	20
2.3. Assessment of the water losses .....	24
2.3.1. Water balance .....	24
2.3.2. Minimum Night Flow .....	27
2.3.3. Performance Indicators .....	28
2.4. Methodologies for the detection and the control of the water leakages .....	31
2.4.1. Leak detection .....	32
2.4.2. Pressure management .....	34
2.4.3. Leakage law .....	36
2.4.4. Active Leakage Control through pressure management .....	37
2.4.5. Sectorization and district metering .....	40
References .....	44

Chapter 3 – OPTIMIZATION OF WATER DISTRIBUTION NETWORKS: A LITERATURE REVIEW .....	49
3.1. Optimal design of network pipes .....	50
3.2. Pump scheduling .....	55
3.3. Pressure management through PRVs and PATs .....	56
3.4. Network segmentation / sectorization .....	59
References .....	62
Chapter 4 – METHODOLOGY .....	69
4.1. Design criteria .....	70
4.2. Problem formulation .....	71
4.2.1. Objective functions .....	71
4.2.2. Constraints .....	76
4.3. Optimization algorithm .....	79
4.3.1. Initialization .....	81
4.3.2. Generation of network partitioning designs .....	86
4.3.3. Floyd-Warshall algorithm .....	91
4.3.4. NSGA-II algorithm .....	93
4.3.5. Hydraulic simulator .....	99
4.4. Technical evaluation of the solutions .....	101
4.5. Example application .....	102
4.5.1. Calibration of NSGA-II .....	105
4.5.2. Algorithm performance evaluation .....	106
4.5.3. Discussion of the results .....	107
References .....	114
Chapter 5 – A REAL CASE STUDY: THE WATER DISTRIBUTION NETWORK OF PIANURA .....	117
5.1. The water distribution network of Pianura .....	117
5.2. Hydraulic modelling of the network .....	123

5.3. Optimal design of DMAs .....	131
5.3.1. Problem setup .....	131
5.3.2. Test cases .....	133
5.3.3. Results and discussion .....	134
References .....	145
Chapter 6 – CONCLUSIONS .....	147
6.1. Limitations of the methodology .....	151
6.2. Future improvements of the research .....	153
6.3. Publications related to this thesis .....	154
Appendix A – EXAMPLE NETWORK TEST PLAN.....	155



## List of figures

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Fig. 2.1 – Water supply system (source: <a href="http://www.pacificwater.org">www.pacificwater.org</a> ).....	11
Fig. 2.2 – Percentages of water losses in the Italian regions (ISTAT 2012) .....	12
Fig. 2.3 – Relationship between domestic water consumption and price in (OECD survey 2007) .....	12
Fig. 2.4 – Trends of water consumption and tariff in (a) Spain and (b) Estonia (EEA 2010) .....	13
Fig. 2.5 – Examples of water leakages ( <a href="http://www.google.com">www.google.com</a> ) .....	16
Fig. 2.6 – Water volumes lost through water leakages.....	17
Fig. 2.7 – Vertical pressure on a buried pipe (Watkins and Anderson 1999).....	18
Fig. 2.8 – Minimum Night Flow in a delimited area (Morrison 2004) .....	27
Fig. 2.9 – Typical layout of a sectorized network (UKWIR 1999).....	40
Fig 4.1 – Fuzzy constraint for water demand in DMAs .....	78
Fig. 4.2 – Schematic of the optimization algorithm .....	79
Fig. 4.3 – Assignment of the network nodes to the DMAs .....	87
Fig. 4.4 – Setting of the boundaries of the DMAs and cascade supply.....	89
Fig. 4.5 – Procedure for opening the best connections between DMAs .....	90
Fig. 4.6 – Floyd-Warshall Algorithm .....	92
Fig. 4.7 – Crowding-distance calculation (Deb et al. 2002) .....	94
Fig. 4.8 – Refinement procedure in NSGA-II (Deb et al. 2002).....	98
Fig. 4.9 – 25-nodes network (Jowitt and Xu, 1990).....	103
Fig. 4.10 – Evaluation of the hyper-volume (Deb 2011).....	106

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Fig. 4.11 – Test case No. 75: (a-e) Pareto fronts obtained in five runs; (f) aggregation of the solutions.....	108
Fig. 4.12 – quasi-Pareto front for the 25-nodes network .....	109
Fig. 4.13 – Optimal solutions for the sectorization of the 25-nodes network ..	110
Fig. 4.14 – 25-nodes network: technical features of the obtained solutions ....	111
Fig. 4.15 – 25-nodes network: closure frequency of pipes .....	113
Fig. 5.1 – Location of the study area (Pianura, Naples, Italy).....	118
Fig. 5.2 – Water distribution network of Pianura: overview and main trunk (a); ground elevations (b) .....	119
Fig. 5.4 – Flow and pressure measurements at the entry point of the network	120
Fig. 5.5 – Examples of pipe removals: (a) dead-trunks; (b) micro-networks with single access (Portolano 2008) .....	123
Fig. 5.6 – Delta-wye transform (Portolano 2008).....	124
Fig. 5.7 – Daily time pattern of the water demands of customers .....	127
Fig. 5.8 – Calibration of the discharge coefficient. Comparison between the detected and the simulated pressures .....	129
Fig. 5.9 – Water pressures in the starting configuration of the network. Peak hour (a); minimum consumption (b).....	130
Fig. 5.10 – Sectorization of the WDN of Pianura: aggregation of the Pareto fronts (first test case).....	134
Fig. 5.11 – Attainment surfaces at different percentiles .....	135
Fig. 5.12 – Pareto front obtained in the second test case .....	136
Fig. 5.13 – Comparison between the EAFs: first (a) and second (b) test cases	138
Fig. 5.14 – Differences between the EAFs: in favor of the first (a) and of the second (b) algorithm .....	138
Fig. 5.15 – Sectorization of the WDN of Pianura: technical plots .....	140
Fig. 5.16 – Sectorization of the WDN of Pianura: closure frequency of pipes	141
Fig. 5.17 – Solution with the minimum value of Total Cost Function .....	142
Fig. 5.18 – Solution with the minimum Resilience Deviation Index.....	143
Fig. 5.19 - Solution with the minimum number of DMAs.....	144

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## List of tables

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Tab. 2.1 – Percentages of losses in the production cycle of drinking water (Portolano 2008).....	19
Tab. 2.2 – Water balance according to the International Water Association (Hirner and Lambert 2000) .....	25
Tab. 4.1 – Network components and parameters required for the optimization.	83
Tab. 4.2 – Unit costs for the 25-nodes network .....	105
Tab. 4.3 – Calibration of NSGA-II. Summary of the tested parameters .....	106
Fig. 5.3 – Lengths of the network pipes classified by diameter (in mm) .....	120
Tab. 5.1 – Total billed consumption for Pianura (years 2007-2009) .....	122
Tab. 5.2 – Sectorization of the WDN of Pianura: network features .....	131
Tab. 5.3 – Sectorization of the WDN of Pianura: analysis options .....	132
Tab. 5.4 – Sectorization of the WDN of Pianura: unit costs.....	132
Tab. 5.5 - Sectorization of the WDN of Pianura: parameters of the NSGA-II (first test case) .....	133
Tab. 5.6 - Sectorization of the WDN of Pianura: parameters of the NSGA-II (second test case).....	133
Tab. 6.1 – Features and payback periods of some remarkable solutions .....	151





### Introduction

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*“As the fundamental and irreplaceable "source of life" for the eco-system, water is a vital good, which belongs to all the inhabitants of the Earth in common. [...] Individual and collective health depends upon it. Agriculture, industry and domestic life are intimately linked to it. Its "unsubstitutable" character means that the whole human community — and each of its members — must have the right of access to water, and in particular, drinking water, in the necessary quantity and quality indispensable to life and economic activity. There is no production of wealth without access to water”.* (Water Manifesto, Committee for the Water Contract: 1998).

In 2010 the United Nations General Assembly agreed upon Resolution A/RES/64/292 which for the first time in history, recognized *“the right to safe and clean drinking water and sanitation as a human right that is essential for the full enjoyment of life and all human rights”*.

Despite these multiple public recognitions of the crucial role of water in our society, in recent years, population growth and the development of non-sustainable human activities have triggered a progressive decrease of the per-capita availability of this resource, thus proliferating its unequal distribution across the globe. Today it has been estimated that over one billion people do not have access to drinking water and that about the 40% of the world population lacks the minimum required amount of this resource to maintain adequate levels of hygiene. In 2004, the British organization “Water Aid” reported that one child died every 15 seconds of diseases contracted due to the scarcity of clean

water, while in 2006 the estimate of the mortality due to water shortage amounted to approximately 30,000 people per day.

The forecasts reported in the first edition of the World Water Development Report (UNESCO, 2003) clearly indicate that in the next twenty years the quantity of water available for each person will be reduced by the 30%. For this reason, in the future an increasingly central role will be assigned to *blue gold* (i.e. water) in social and economic policies, and will continue to define the relationships between different countries.

Obviously, the problem varies in different areas of the world. In the arid and poorest regions, water scarcity (accentuated by climate change due to the greenhouse effect) is dramatically worsened by the shortage of infrastructure for water abstraction and supply. Conversely, in industrialized areas increasing water demand due to population growth and the development of economic activities collides with a non-sustainable use of the natural resource, resulting in the following:

- progressive depletion of the aquifers and reduced availability of new water sources for pollution-related phenomena;
- significant waste of water due to non-rational consumption and huge amounts of water losses in supply systems and distribution networks.

This gives rise to the need for innovative policies based on rationalization and the economy which steps through the development of a real “culture of the water”, together with an appropriate planning of the use of resources and an adequate management of water services.

In this context, a significant contribution is provided by the regulatory framework recently introduced at both the national and international level, which aims at improving the quality of public utilities management through the attainment of specific objectives, whose features are measurable and comparable over time and space. This new legislation stresses the crucial role played by the

prevention and the reduction of leakages in water distribution networks, being among the main goals that must be achieved by the water companies.

Water systems worldwide are suffering from significant inefficiencies caused by inadequate management strategies. For example, Italy has an extremely fragmented water network, which is characterized by an overall length of 210,000 km and a generally unsatisfactory level of conservation. The outcomes of a recent survey conducted by the Italian authority for the supervision of the water resources (Commissione Nazionale di Vigilanza sulle Risorse Idriche, CoNViRI, today Autorità per l'Energia Elettrica, il Gas ed il Sistema Idrico, AEEGSI) show that the water losses at a regional scale vary in the range between 21% and 61%, with an average national value of 37% (CoNViRI, 2009). This alarming scenario has several implications:

- *social detriment* due to the disadvantages that may be caused by the possible scarcity of supply;
- *economic damages* related to the loss of huge volumes of clean water, whose production often requires high cost (for abstraction, treatment, pumping, etc.), and to the required investment for the research and the use of supplementary sources;
- *environmental effects* caused by the subtraction of resources from the natural cycle and the alteration of the underground water flows in the laying soil of the networks.

### **1.1. Problem statement**

The mitigation of the water losses should primarily be achieved through a proper maintenance of the systems. However, the recovery of a sufficient degree of efficiency of the network pipes may result in very high costs that are often conflicting with the operating budgets, especially in highly urbanized areas.

On the other hand, the increased attention on this topic has led to the development of innovative strategies for the control and the reduction of the water leakages. Such methodologies, which are mainly based on the smart management of water pressures, consist of interventions with low economic impact that are able to improve the operation of the water network as a whole.

Among these techniques, *sectorization* makes it possible to gain further control of this complex component of the water supply system. Nevertheless, the advantages offered by this approach clash with the difficulties in planning and implementing the possible solutions. Actually, in common practice this activity is mostly delegated to the technicians involved in the operational management of the network itself. However, at the engineering level a significant contribution to the efficiency of this kind of analysis is provided by the availability of Decision Support Systems (DSSs).

## **1.2. Purpose of the work**

The aim of the present thesis is that of introducing an innovative approach for the sectorization of the water distribution networks. The main goal of the presented methodology is that of optimizing the design of the network partitioning in order to achieve: i) a proper control on the system; ii) the minimization of the operational cost; iii) the preservation of the hydraulic performance.

The developed method follows a multi-criteria and multi-objective approach, in which particular attention is paid to the mitigation of water leakages. An optimization algorithm is defined, which is able to provide feasible solutions to the addressed problem in a completely automatic way, starting from the available information about the water distribution network.

### 1.3. Outline of the thesis

The thesis is structured as follows:

- **Chapter 1** is this brief introduction;
- **Chapter 2** discusses the generalities about the management of water supply systems, and in particular of the water distribution networks. It also focuses on the definition and the characterization of the water losses, which represent one of the main problems that affect this kind of infrastructure;
- **Chapter 3** presents a review of the literature studies that have been carried out in the addressed field of research. The description of the state of the art in research concerning the single and multi-objective optimization of the water distribution networks is useful to outline the specific context of the new methodology that is introduced here. In particular, the discussion of the approaches developed so far about the sectorization provides further insight into the motivation of the present work;
- **Chapter 4** consists of a detailed description of an innovative technique for the automatic and multi-objective sectorization of a water distribution network. The mathematical formulation of the problem and the characterization of the different steps of the designed procedure are provided here. The basic algorithms and tools included in the methodology are described as well. In addition, the application to a very simple example from the literature is presented with the aim of providing further details about the developed approach. The obtained results are thoroughly discussed and are then used as the basis for a subsequent analysis;
- **Chapter 5** shows the outcomes of the optimization of a real-world network carried out with the proposed methodology. The input data and the hydraulic modeling are introduced and discussed, as well as the basic assumptions of

the performed analysis. An in-depth overview of the technical features of the provided solutions is presented in order to highlight the potential of the achievable results;

- **Chapter 6** summarizes the content of the thesis and the contribution to the scientific research in the addressed field of research. It also points out the main limitations of the methodology and of the optimization algorithm, and presents possible future improvements.

## References

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# Management of water systems

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Water supply systems are crucial infrastructures that provide water to city centres, industries, rural villages and, where possible, to single buildings. Even though their construction started in ancient times, their use was largely spread by the Romans, who introduced high qualitative and technological standards that lasted nearly unrivalled for over thousand years after the fall of the Western Roman Empire.

In the late Renaissance, further progress in this field started throughout Europe and in particular in Italy, France and England. In the 18<sup>th</sup> century, Luigi Vanvitelli designed the Caroline Aqueduct to convey the water from Mount Taburno to the Royal Palace of Caserta across a 38 km route. In the same period, rapid population growth in London and in other English cities (Birmingham, Liverpool and Manchester) fuelled a boom in the establishment of private and public water supply networks.

The largest examples of water supply systems can be found in the United States, where a huge amount of infrastructure is needed in order to provide water to their huge cities. Apart from the Catskill Aqueduct, which supplies part of the city of New York with an estimated capacity of 1,500,000 m<sup>3</sup> of water per day, the most impressive is probably the Colorado River Aqueduct, a 389 km long system composed of canals, tunnels, buried conduits and siphons, which conveys drinking water from the Colorado River to the Southern California.

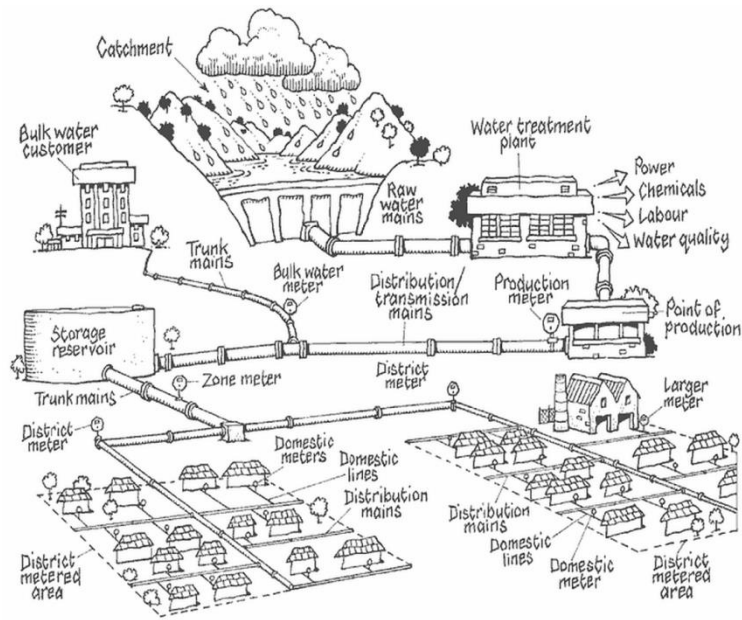
The development of pipeline technology with increased resistance and versatility brought about the introduction of pressurized water supply systems. In addition to the benefits related to the hydraulic operation, such plants are able

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to ensure the best performance in terms of protection of the water quality. Actually, the main technical issue in the design of the water infrastructures throughout history has gradually shifted from the mere supply to the safety of the public health. This problem assumed a remarkable social importance especially with the advent of industrialization and the consequent unprecedented urban development.

However, as noted before, even today there are many examples of aqueducts characterized by open-channel flows, in particular for the main supply lines. Regardless of the hydraulic operation, a modern aqueduct for the drinking water is made of the following components:

- **Water intakes**, that allow the abstraction of water from the natural cycle. Their features depend on whether the water is collected from surface sources (rivers, lakes) or aquifers (underground flows).
- **Treatment plants**, located immediately downstream of the water intakes, and whose function is the removal of undesirable contaminants in order to produce water that fits specific uses (human consumption, industrial activities, etc.);
- **Transmission mains**, through which water collected at the source is conveyed in the proximity of the end users;
- **Storage facilities** (tanks), whose dimensions must be suitable to provide adequate reserves to cope with the disruptions of the transport lines and to provide the integration of the flow rates required for meeting the water demand during the peak hours;
- **Distribution networks**, which are typically characterized by looped layouts and are meant to deliver water to all the customers.



**Fig. 2.1 – Water supply system (source: [www.pacificwater.org](http://www.pacificwater.org))**

The management of such complex systems is one of the most challenging problems in engineering practice. Maintenance of the water distribution networks has gained particular importance in the light of the increasing attention on environmental issues and the resulting increase in the adoption of sustainable development strategies. Nowadays, the problem of greatest interest is probably the improvement of the efficiency of the water distribution networks, particularly with regard to the reduction of the energy consumption and the waste of a key natural resource due to the water losses.

Actually, in many contexts the conservation status of the networks raises several concerns. For example, the latest report provided by the Italian National Institute of Statistics (Istituto nazionale di statistica, ISTAT) shows that the percentage of total water losses in the Italian regions (Fig. 2.2) varies between 21.9% (Aosta Valley) and 54.8% (Sardinia), with a national average value of 37.4% (ISTAT 2012). The amount of lost water would be enough to meet the demand of over the 35% of the current population, a fact which demonstrates the true severity of the situation.

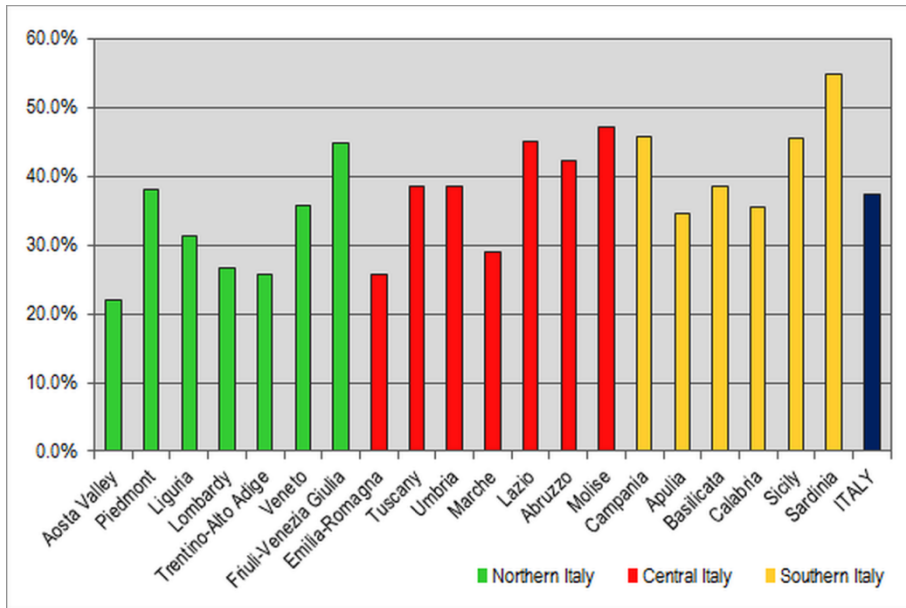


Fig. 2.2 – Percentages of water losses in the Italian regions (ISTAT 2012)

At the same time, it must be highlighted that the water pricing in some countries is not efficient in encouraging the sustainable use of this natural resource, especially at the level of the end users. Actually, in recent years several international surveys have shown that in the most developed and wealthy countries the price of water is inversely correlated with the per-capita consumption (Fig. 2.3).

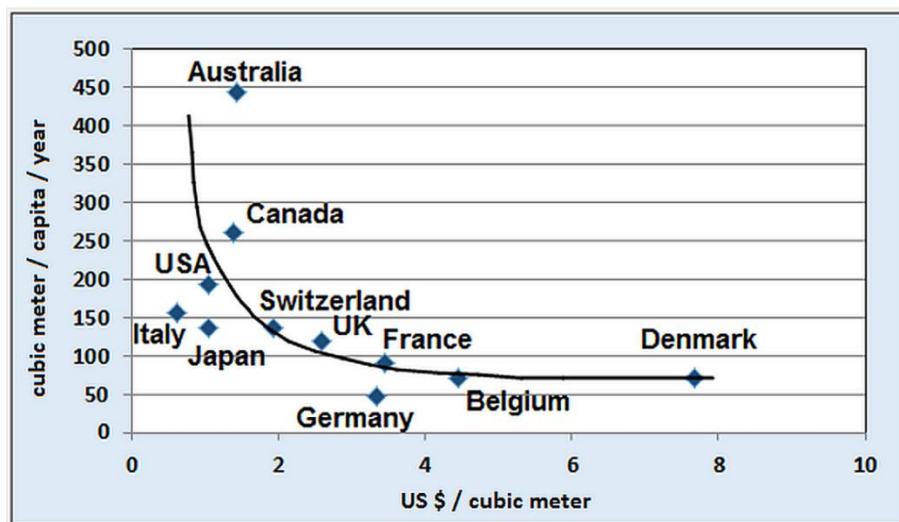
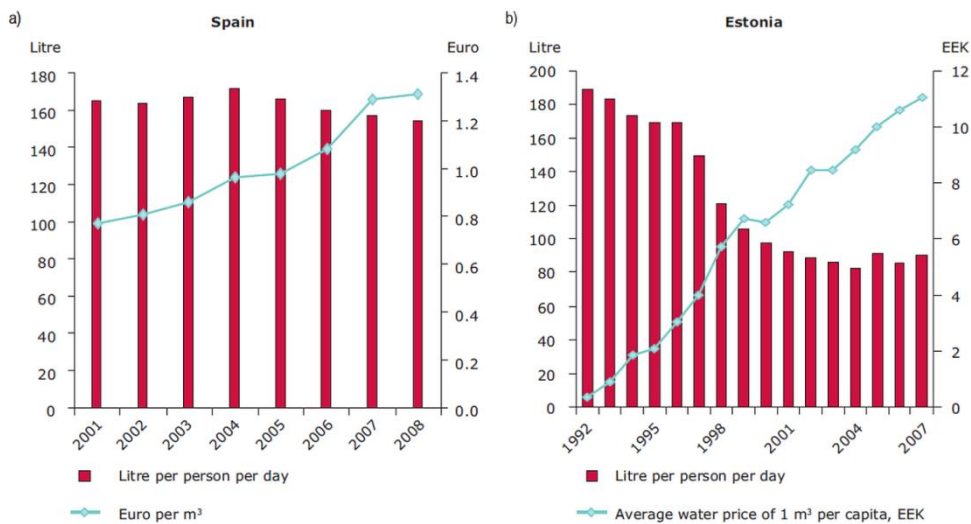


Fig. 2.3 – Relationship between domestic water consumption and price in (OECD survey 2007)

Nevertheless, in recent years a significant increase in the cost of drinking water has been observed on a global scale. In some contexts this situation has led to a reduction in consumption (Fig. 2.4), as highlighted in studies carried out in Spain and Estonia (European Environment Agency, EEA 2010).

However, even though this policy is intended to promote a more stringent use of water, it inevitably reflects the budget shortfalls caused by the inefficient management of the networks. In this regard, particular attention must be paid in the light of the very important social role played by water. One of the most important objectives to achieve in the management of supply systems should be that of providing an affordable access to water to those in lower-income groups (Organization for Economic Co-operation and Development, OECD 2009).



**Fig. 2.4 – Trends of water consumption and tariff in (a) Spain and (b) Estonia (EEA 2010)**

The improvement of the efficiency of the water networks is also related to the protection of the environment. The production of drinking water involves an environmental burden that goes far beyond the mere abstraction of water volumes from natural ecosystems. Some examples are the energy required by the pumping stations and the pollutant load of the waste generated by the water treatment.

The issues described so far have been recently addressed with strong emphasis by the legislative bodies and the international scientific community, with a strong focus on improving the knowledge about these topics and on the development of strategies for the optimization of the water systems. These aspects are discussed in more detail later in the text. A preliminary discussion about the water losses is provided in the following section.

## **2.1. Water losses in supply and distribution systems**

The term “water loss” is generally adopted to indicate the difference between the overall amount of water supplied in the network and the sum of the water volumes corresponding to the customer consumption recorded by the flow-meters.

This definition makes it possible to highlight the different meaning of the “waste of water”, that is mainly related to the negligent use of this resource (e.g. taps left open accidentally) and to the inefficiency of the household plumbing systems, whose status is not always monitored with proper attention.

These water losses can be divided into two groups: the *apparent losses*, which consist of water volumes actually consumed but not accounted for, and the *real losses*, that are caused by large damages that may have occurred to the network pipes or by the deterioration of the pipe junctions or the hydraulic devices. Some examples of apparent losses are listed below:

- unrecorded water volumes used for public functions, (e.g. cleaning of roads and urban areas, irrigation of green spaces, operation of public fountains, fire-extinguishing service);
- service volumes required for the proper operation of the water service (e.g. cleaning of pipes and tanks after maintenance interventions);

- water losses due to the improper operation of the network plants (e.g. unnecessary opening of exhaust valves);
- authorized consumption affected by measurement errors (e.g. inaccuracy of flow-meters, data handling errors in customer billing systems);
- unauthorized withdrawal of water volumes from the network (e.g. illegal connections, tampering of the flow-meters).

These losses represent on average about 30% of the total losses, and they hold considerable importance from both the financial and the social points of view. The assessment of the actual amount of apparent losses is a very challenging problem that can only be tackled through an in-depth knowledge of the assets and the operation of the water network itself.

Real losses are the physical losses of water from the distribution system, also referred as “water leakages”. These losses put a strain on water supply and inflate the management cost for the water utilities since they represent water that is extracted and treated but never reaches the end users.

In many cases, minor water leakages deriving from the inefficient hydraulic seal of junctions or from small cracks on pipes may lie hidden for a long time, sometimes for months or even years. Major leakages can be easily observed when significant damages to the pipes occur, as they usually result in large amounts of water erupting from ground or flowing in the customer properties.

According to the BABE (Burst And Background Estimate) method (Lambert 1994), the real losses (Fig. 2.5) can be classified into the following categories:

- *background losses*, namely the aforementioned minor water leakages, that cannot be discovered without visual inspection. In common practice, the real water losses that are identified as background losses are those for which the dispersed flow at the operating pressure of 50 m does not exceed the limit of 500 litres per hour;

- *reported bursts*, which correspond to huge failures of pipes and that are easily revealed by the significant damages that they are likely to cause to roads, buildings and other infrastructure. Since they can be identified relatively quickly, these water leakages usually have a short lifespan;
- *unreported bursts*, which have a reduced magnitude compared to reported bursts, and whose detection is usually carried out through instrumental campaigns that are mainly based on the produced noise.



*Fig. 2.5 – Examples of water leakages (www.google.com)*

Another point of interest is the water volume lost through a single leakage. In the BABE method, this is calculated as the product between the burst flow rate and the average duration, which is made of three main components (Fig. 2.6): the *time for awareness* ( $A$ ), that is the duration between the first occurrence of the burst and the water utility awareness of its existence; the *time for location* ( $L$ ), which is the time required for detecting the precise location of the leakage; the *time for repair* ( $R$ ), that is required for the rehabilitation of the leaking pipe. As shown in Fig. 2.6, large flow rates (like those observed in reported bursts) are associated to short durations, and vice-versa.



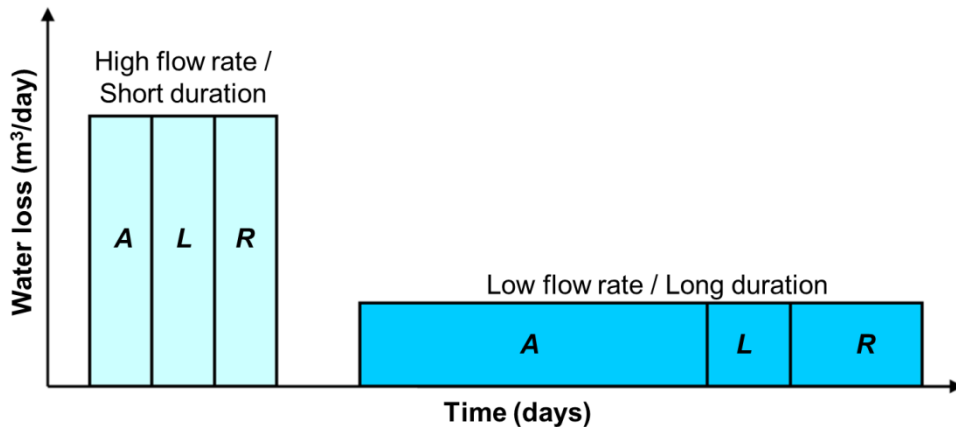


Fig. 2.6 – Water volumes lost through water leakages

There are many factors that contribute to the generation of the real losses. Among these, the mechanical properties of the laying soils should be taken into account, as their electrochemical interaction with the pipes influences the frequency of the pipe bursts over time and space. This also depends on the technology adopted for the building of the water networks, both in terms of construction techniques and pipe materials.

The conservation of the water network over time is also conditioned by the length of the pipes and the number of pipe fittings and regulating devices (e.g. valves), as these determine the number of required junctions and, therefore, the system vulnerability.

Similar considerations can be made about the frequency of the customer connections. In addition, it should be emphasized that their construction is not always performed with sufficient expertise. Moreover, their underground depths are usually much less than those of the main trunks, and consequently they are more exposed to the stresses resulting from vehicular traffic.

Figure 2.7 shows the typical trends of the vertical pressure on a buried pipe at different cover depths. The contributions of both the soil and the traffic loads are presented as well. It can be clearly observed that while the former increases with the cover depth, the latter decreases to zero asymptotically.

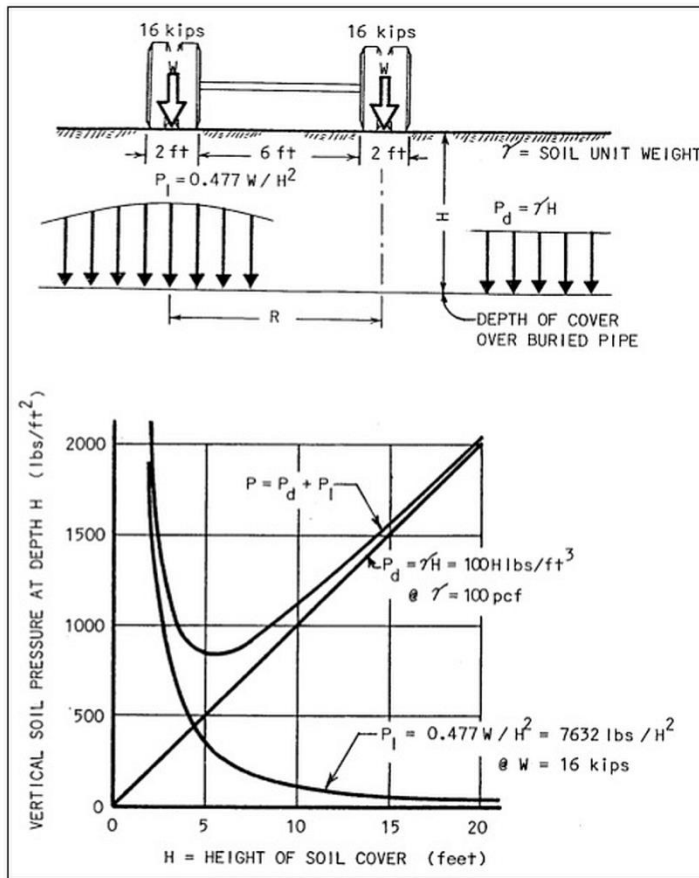


Fig. 2.7 – Vertical pressure on a buried pipe (Watkins and Anderson 1999)

The maintenance policy adopted in the management of the water networks also influences the magnitude and the occurrence of the real losses. In this regard, the overall age of the pipelines is a crucial aspect, as well as the frequency of the interventions for the rehabilitation or the replacement of the pipes.

In most cases, the strategy operated by the drinking water utilities around the world is that of focusing on the reported bursts, while less investments are allocated to the detection of hidden leaks and in proactive programs for the mitigation and the prevention of background losses. However, utilities that employ such reactive responses to this phenomenon are likely to have excessive levels of water leakages that cannot be contained with sufficient reliability (American Water Works Association, AWWA 2009).

After all, several technologies and strategies have been developed in recent years that allow the water utilities to identify, measure, reduce or eliminate leaks with expenditures that are consistent with the budget requirements. These cost-effective techniques include: i) the analysis of the flows and components to quantify the amounts of real losses; ii) noise correlators and loggers to pinpoint the pipe bursts; iii) pressure management to systematically reduce the water leakages in both the short and the long terms. A more detailed discussion of this is provided later in this chapter.

In general, the real losses can be estimated in approximately 70% of the total losses. Even though they affect all the production cycle of the drinking water, they are mainly located in the distribution networks, as reported in the following table:

<b>Water system components</b>	<b>Water losses (%)</b>
Water intakes	2
Treatment plants and transmission mains	15
Storage facilities	4
Distribution networks and customer connections	20

*Tab. 2.1 – Percentages of losses in the production cycle of drinking water (Portolano 2008)*

These significant amounts of water losses can also be found in the water networks with the best levels of conservation. As widely reported in the scientific literature, there is a minimum limit of physiological water losses that cannot be overcome, and that can be estimated through the *UARL* (Unavoidable Annual Real Losses) index (Lambert et al. 1999):

$$UARL = (18 \times L_m + 0.8 \times N_c + 25 \times L_p) \times P \quad (2.1)$$

where *UARL* is expressed in litres/day,  $L_m$  (in kilometres) is the overall length of the network mains,  $N_c$  is the number of customer connections,  $L_p$  (in kilometres) is the average length of the private pipes (between the street property boundary and the customer flow-meter), and  $P$  is the average operating pressure (in

metres). This equation, based on the component analysis of real losses for well-managed systems with good infrastructure, has proved to be robust in a variety of different international contexts (Lambert and McKenzie, 2002). Actually, *UARL* is the most reliable predictor of the minimum achievable level of real losses for systems with more than 5,000 service connections, connection density ( $N_c/L_m$ ) over 20 per km, and average pressure larger than 25 metres.

Therefore, from a financial perspective if the water demand is adequately satisfied it is not convenient to push the efforts for the mitigation of the water leakages over this given limit. Indeed, when the percentage of losses is between 5% and 10% of the overall water intake, the saving that could be achieved by reducing the water leakages would not compensate the required investment.

However, as discussed before, it is not possible to approach this issue solely in economic terms. In reality, water is a vital natural resource that should be protected regardless of its availability. In recent years, this concept has been strongly addressed by the Italian regulatory framework, whose main features are briefly described in the next section.

## **2.2. Management of water networks in the Italian legislation**

Over the last three decades, increased sensitivity to environmental issues has determined a considerable enrichment of the Italian legislation related to this topic. Consistent with international guidelines, several regulatory actions have been undertaken with the aim of improving the rational use of the natural resources, including drinking water.

The first normative reference in this regard is represented by the national master plan of water systems (*Piano Regolatore Generale degli Acquedotti*, PRGA, L.129/63), which was developed during the 1960s in order to regulate the water supply throughout the Italian territory. The preparation of the PRGA was based on field studies carried out up to the year 1961 about the per-capita water consumption and the resident and fluctuating population in every Italian

municipality. The results of these studies made it possible to predict the evolutionary scenarios of the water demands, and in particular their estimate at the year 2015. In addition, the localization of new possible water sources allowed for a rational planning of the required interventions for fulfilling the new demands and for the integration of the pre-existing ones.

General guidelines about the environmental protection were issued in 1976 with the “Merli’s act” (“*Norme per la tutela delle acque dall’inquinamento*”, L. 319/76), while more significant innovations were introduced in 1989 with the law named “*Norme per il riassetto organizzativo e funzionale della difesa del suolo*” (L. 183/89). Among other things, this law promoted the establishment of the first territorial authorities for the management of the water resources (*Autorità di Bacino*).

However, the most important changes were provided in 1994 by the framework law No. 36, also known as the “Galli’s act”, which revolutionised the whole water management cycle, from its abstraction to its return to the environment. This law was born with the aim of solving the excessive fragmentation of the Italian water utility system. Actually, in 1987 the ISTAT attested the presence of 6,200 companies for the drinking water service, 7,000 for waste water management, and over 2,000 for water purification. The salient points of the Galli’s act are reported below:

- the introduction of the concept of “integrated water service” (*Servizio Idrico Integrato*, SII), for which all the steps of the technological cycle of urban water must be included in a unique management;
- the establishment of local authorities called *Ambiti Territoriali Ottimali* (ATOs) in charge of the organization of the SII at the hydrographic scale. In practice, the jurisdiction area of each authority is not drawn on the basis of administrative boundaries but according to the geographical and environmental criteria;

- the adoption of the *water balance* (see Sec. 2.3.1) as fundamental instrument for the identification of quantitative imbalances between the availability and the use of the water resources;
- the introduction of industrial criteria in the organization of the SII, which should be oriented to the achievement of a balanced economic and financial management. To this aim, the separation between the holders of the service (the ATOs) and the management companies (water utilities) is prescribed;
- the regulation of the criteria for the determination of the water tariff, that must be consistent with the cost of the service and the required investments. This evaluation should be carried out at local scale on the basis of the guidelines provided by the Ministry of Public Works (today, the Ministry of Infrastructures and Transports);
- the characterization of the *minimum service requirements* for the water supply. In particular, the customers must be provided with a continuous supply (24/7, except in extraordinary cases or planned maintenance operations) corresponding to: i) a per-capita availability of at least 150 liters/inhabitant/day; ii) a minimum discharge of 0.1 lps at the delivery points of the households; iii) a minimum pressure of five meters above the top floor of the building located at the highest elevation in the network; iv) a maximum pressure of 70 meters above the roadway at the delivery points.

The Galli's act has been almost completely abrogated by the legislative decree No. 152/06, entitled "*Norme in materia ambientale*" (best known as "*Testo Unico Ambientale*"). However, the third section of the decree resumes with a similar approach to the regulation of the water resources management, and it strongly emphasizes the public property and the inalienability of water infrastructure granted in concession with water utilities. In addition, it promotes new concepts to adopt in the management of the SII, that should be carried out under the criteria of *effectiveness, efficiency and cheapness*.

With regard to the specific legislation about the water losses, a first example can be found in the Prime Ministerial Decree dated 4<sup>th</sup> March 1996, which is the implementation decree of the Galli's act. In particular, the decree stated that:

*“[...] l'attività di gestione deve tra l'altro garantire il risparmio idrico, attraverso l'adozione di misure mirate alla riduzione delle perdite in rete, al recupero dell'acqua non contabilizzata, al contenimento degli sprechi ed alla gestione della domanda in condizioni di scarsità della risorsa idrica [...]”.*

*“[...] the water utility should be able to ensure the water conservation through the adoption of actions aimed at the mitigation of the leakages, the reduction of the unbilled consumption, the limitation of the waste of water and the proper management of the demand when shortage of resource occurs [...]” ).*

At the same time, consistent with the high inefficiency of the existing networks, the decree prescribed the need for specific intervention only in cases with water losses above the 20% of the total system volume, which was the upper limit for the technical acceptability of the water leakages.

Nowadays, the most advanced normative reference is the Ministerial Decree No. 97/99 (*“Regolamento sui criteri e sul metodo in base ai quali valutare le perdite degli acquedotti e delle fognature”*), which introduced the obligation on the part of the water utility companies to organise and monitor the water networks. Also in this case, particular attention is paid to the estimation of the water balance of the network according to the flow measurements performed in a given period. On this basis, water utility companies are made to carry out the localization of the water leakages and the rehabilitation of the network pipes.

The decree also focused on the nature and on the possible causes of the water losses, and it stressed the need to adopt advanced and pro-active strategies for the mitigation of those water leakages that cannot be completely detected, even through proper monitoring.

### **2.3. Assessment of the water losses**

The implementation of the criteria of effectiveness and economy in the management and the maintenance of the water systems strictly requires a reliable estimate of the water losses. This is a key factor allowing for the identification of the areas affected by higher vulnerability, and for the planning of the required actions for the recovery of the water losses with sustainable expenditures.

In this regard, the most successful and generally adopted methods are the estimation of the water balance of the network (*Top-Down* approach) and the detection of the Minimum Night Flow (*Bottom-Up* technique). Additional information can be provided through the evaluation of *Performance Indicators* related to the system operation.

#### **2.3.1. Water balance**

The characterization of the water balance primarily involves an accurate knowledge of the water system and its assets, that is required for the quantification of the input and output flows relevant to every element of the network.

Furthermore, in order to make the comparison between different contexts and to improve the readability of the results, the definition of the components of the water balance should be carried out with reference to acknowledged standards. To this aim, the approach introduced by the International Water Association (IWA) is presented here, as it is the most commonly adopted in both the technical and scientific communities worldwide.

Table 2.2 shows the full breakdown of the IWA water balance (Hirner and Lambert 2000). As highlighted by the same authors, some difficulties may be experienced in its characterization when sufficient information is not available. However, the separation of Non-Revenue Water into components (Unbilled



Authorised Consumption, Apparent Losses and Real Losses) should always be attempted.

System Input Volume	Authorized Consumption	Billed Authorized Consumption	Billed Metered Consumption	Revenue Water
			Billed Unmetered Consumption	
		Unbilled Authorized Consumption	Unbilled Metered Consumption	Non-Revenue Water
			Unbilled Unmetered Consumption	
	Water Losses	Apparent Losses	Unauthorized Consumption	
			Metering Inaccuracies	
		Real Losses	Leakage on Transmission and/or Distribution Mains	
			Leakage and Overflows at Utility's Storage Tanks	
			Leakage on Service Connections up to point of Customer Metering	

*Tab. 2.2 – Water balance according to the International Water Association (Hirner and Lambert 2000)*

All the entries should be provided in terms of water volume/year. A more detailed description of the components is reported below:

- *System Input Volume*: is the annual input to a defined part of the water supply system;
- *Authorized Consumption*: the annual volume of metered and/or non-metered water taken by registered customers, the water supplier and others implicitly or explicitly authorised to do so. It includes the exported water and leaks and overflows after the point of customer metering, as well as the public uses of water (fire fighting, flushing of mains and sewers, street cleaning, watering of municipal gardens, operation of fountains, etc.). These may be billed or unbilled, metered or unmetered according to local practice;

- *Water Losses*: the difference between System Input Volume and Authorised Consumption, consisting of Apparent Losses and Real Losses. They can be considered as a total volume for the whole system, or for partial systems such as water mains, transmission or distribution;
- *Apparent Losses*: they consist of unauthorized consumption (theft or illegal use), and all types of inaccuracies associated with production metering and customer metering. Under-registration of production meters and over-registration of customer meters lead to the under-estimation of the real losses, and vice-versa;
- *Real Losses*: the physical water losses from the pressurised system, up to the point of customer metering. The volume lost through all types of leaks, bursts and overflows depends on frequencies, flow rates and average durations of individual leaks.

From a purely economic point of view, the System Input System can be considered as the sum of the following rates:

- *Revenue Water*: water volume that actually produces revenue for the water utility company, and that is clearly equal to the Billed Authorized Consumption;
- *Non-Revenue Water*: the sum of the Water Losses and of the Unbilled Authorized Consumption, that represents the share of the production that is not paid back and on which the water utility should focus the efforts for improving the efficiency.

### 2.3.2. Minimum Night Flow

The Bottom-Up approach consists of the estimate of the water losses according to the minimum flow measurements performed during the hours of minimum consumption (Minimum Night Flow, MNF). This methodology is based on the following assumption: even though there is always a legitimate demand for water at night, the bulk of the detected flows is surely attributable to the water leakages, whose amount in this period is increased because of the higher pressures that are established in the network.

The assessment of the MNF is usually carried out on hydraulically isolated zones of the network (see Sec. 2.4.5) between the hours 1:00 and 4:00 am (Ratnayaka et al. 2009). The continuous sampling of the input flows is typically achieved through the use of flow-meters connected to *data loggers*. A typical trend of the MNF over time (in days) is reported in Fig. 2.8, that clearly shows the high variability related to the pipe bursts.

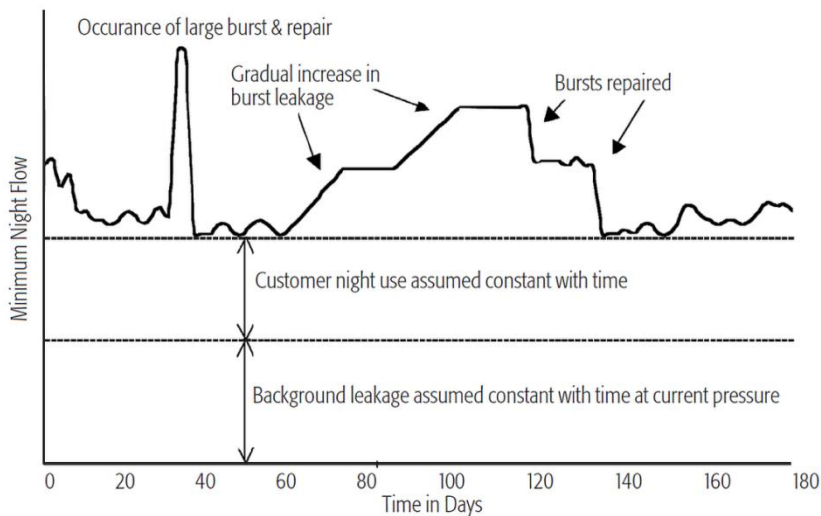


Fig. 2.8 – Minimum Night Flow in a delimited area (Morrison 2004)

Although MNF is not a direct measure of the quantity of lost water, it can be considered one of the most reliable indicators of the conservation level of the network. However, significant inaccuracies may affect the estimate of the real

amount of water losses when the characteristics of the investigated areas are not properly taken into account.

For example, this is the case in residential buildings equipped with large private storages that are filled at night. In addition, particular attention should be paid to the non-domestic night consumption (e.g. presence of hospitals, industrial night productions, high density of bars and nightclubs) that may have a significant impact on the features and the occurrence of the MNF.

Therefore, it is important to compare the detected measures with reference values that are estimated according to the extent of the investigated area, the type of customers, the time of the year during which the measurements are performed.

### **2.3.3. Performance Indicators**

The assessment of the efficiency of the water systems can be boosted through the evaluation of a number of Performance Indicators (PIs), which are able to provide clear and synthetic information about the operation of the water networks. Furthermore, they make it possible to compare different management contexts and to identify target levels of performance for water utilities to aim for.

In this regard, one of the most important references in the scientific literature is the “IWA Manual of Best Practice” (Alegre et al. 2000). In this document, 133 different indicators are described that are related not only to the water leakages, but more in general to the quality, the economy and the operation of the water service.

However, with regard to the water losses, several indicators are introduced at different levels. The first group (*level one indicators*, L1) includes very simple indices that are commonly adopted in different countries worldwide in order to pan the overall status of the water system. Among these indicators, the most used are probably the following:

- *Percentage by volume*, which consists of the per cent ratio between the water losses and the system input volume. This is the most traditional and widely adopted indicator in the technical literature because of the simplicity of its evaluation and understanding;
- *Water loss per customer connection*, whose evaluation is complicated by the extreme uncertainty about the number of connections in the distribution network (especially in Italy);
- *Water losses per unit length of network pipes*, that is particularly interesting for the estimate of the water losses in rural areas, where the number of customer connections per kilometre is less than twenty.

Another key indicator for the assessment of the reliability of the water system is the *mechanical failure rate* ( $\lambda$ ), whose definition is reported below:

$$\lambda = \frac{N_F}{L} \quad (2.2)$$

where  $N_F$  is the annual number of pipe failures (which can be estimated through the number of extraordinary maintenance interventions) and  $L$  is the overall length of the network pipes expressed in kilometres. There are many factors that contribute to the mechanical failure rate, most of them relevant to the specific context under examination. Literature studies have reported very different values of  $\lambda$ , which ranges from a minimum of 0.05 failures/km/year to a maximum of 1.00. However, several relationships have been introduced according to the general trends detected in relation to: the pipe age (Shamir and Howard 1979); the pipe diameter (Kettler and Goulter 1983; Su et al. 1987; Goulter and Kazemi 1988, 1989; Mays 1989; Cullinane et al. 1992; Goulter et al. 1993); both of the previous (Kettler and Goulter 1983; Giustolisi et al. 2006); the climatic conditions (Harada 1998; Welter 2001; Ahn et al. 2005).

A more detailed characterization of the network operation is made possible by the evaluation of more complex indicators (*level three indicators*, L3). In addition to the already mentioned *UARL* (see Sec. 2.1), other examples are the *CARL* (Current Annual Real Losses) and the *ILI* (Infrastructure Leakage Index). The former is the actual amount of physical losses in the considered year, while the latter is given by the ratio between the *CARL* and the *UARL* (Lambert et al. 1999).

The *ILI* measures how effectively a utility is managing real losses under the current operating pressure regime. For example, an *ILI* equal to three means that:

- the physical losses in the network are three times higher than the unavoidable losses;
- the real losses can be reduced by one-third at equal pressure;
- a further reduction of the water losses can be achieved through the modification of the pressure regime.

Instead, small values of the *ILI* (close to one) are associated to the best performance of the network management. However, the pursuit of such objective is justified only in cases of scarcity of resource or when the marginal cost of the water is particularly high.

## **2.4. Methodologies for the detection and the control of the water leakages**

Before analysing the different techniques that are commonly adopted for tackling the real losses, it is important to emphasize the need for a thorough knowledge of the structure and the operation of the water systems. In this regard, a significant contribution is offered by the availability of management tools and data storage, such as a *Geographic Information System (GIS)*.

In particular, a GIS software makes it is easier to handle the huge amount of information that is required for the proper operation of the networks. To this aim, the stored data should consist at least of the following (Giugni et al. 2003):

- typology, material and age of the pipes;
- mechanical and chemical properties of the laying soils;
- physical phenomena affecting the network pipes (with particular reference to corrosion and tuberculation);
- service ineffectiveness issues (e.g. water pressure below the minimum level);
- location, features and operation of pumps and valves;
- detailed description of the repair / replacement interventions on the network pipes and devices (and relevant costs);
- water quality;
- customer claims.

In addition, the GIS software allows the decision maker to carry out spatial analysis operations that provide very useful indications for the planning of the future activities. For example, a geo-referenced database of the performed maintenance interventions makes it possible to identify the areas characterized by higher vulnerability on which the water utility should focus the improvement works (Bertola and Pavia 2002).

The approaches that are commonly adopted for the management of the water leakages can be categorised into two main groups:

- strategies based on Passive Leakage Control (*reactive control*);
- strategies based on Active Leakage Control (*proactive control*).

The first category basically indicates the approach of reacting to visible leakages due to pipe bursts and that are usually discovered by customers or by the staff of the water utility through direct observation, unexpected drops in pressure or secondary effects generated on buildings and other structures. Obviously, this policy reduces the cost of the system maintenance in the short-term period, but it is associated to the highest inefficiency in the management of the water networks.

The proactive control refers to procedures taken by the water utilities (with special team of dedicated staff) to monitor, repair and maintain the leakage level as a regular activity. This includes the following:

- leak detection through regular survey;
- pressure management;
- district metering.

#### **2.4.1. Leak detection**

The *regular survey* consists of the planned and systematic research of the water leakages through experimental campaigns that can be based on different techniques. Among these, the most commonly adopted are the *acoustic methods*, which aim at discovering and locating the leakages through the detection of the sound waves produced by the outgoing water from a crack. These waves can be distinguished into two types: i) waves that propagate longitudinally along the pipe; ii) waves that propagate almost radially into the surrounding soil.



There are many factors that affect the characteristics of the sound waves, such as the water pressure, the material and the diameter of the pipes, the type and the saturation of the laying soils, the presence of joints and discontinuities. *Listening rods*, *ground microphones* (also known as “geophones”), *noise loggers* and *noise correlators* are used for detecting the sound. The noise frequency in proximity of the leak is usually between 20 and 250 Hz, while the human listening is limited to about 50 Hz. Therefore, some of these devices can be equipped with amplifiers in order to allow the operators to hear the low frequency noise. The same issue also represents the main disadvantage in the adoption of the acoustic method, as its effectiveness relies upon the experience and the sensitivity of the operator (Mutikanga 2012).

Other non-acoustic technologies have been developed for discovering the water leakages: the *optical method*, which consists of the inner inspection of the pipes through special CCTV cameras; the *tracer gas method*, which involves the filling of the pipelines with a mixture of air and non-toxic gases (mainly helium and hydrogen) that escapes from the cracks and rises to the ground surface, where they are detected by mobile sensors; the *infrared thermography method*, that is based on the detection of the anomalies in the ground surface temperature generated by the presence of water or large voids in the first layer of the laying soil of the pipes; the *ground penetrating radar method*, which measures the reduction of the celerity (i.e. the velocity of propagation) of the radar waves that is caused by the water coming out from a leak. Although these technologies have been found to be promising, they are not used on a wide scale because of the high associated costs, the difficulties in understanding the results and some disadvantages determined by their application (e.g. the optical and the tracer gas method require the preliminary emptying of the pipes).

Another interesting approach is the leak detection method based on the analysis of the hydraulic transients (Brunone 1999; Mpesha et al. 2001; Wang et al. 2002; Ferrante and Brunone 2003; Covas et al. 2005; Lee et al. 2005). This is a non-intrusive technique that consists of the generation of controlled transients

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that can be achieved, for example, through the opening/closure of network valves or fire hydrants. The presence of a leak creates a partial reflection of the elastic waves that is used to estimate the location and the size of the leak itself. However, although justified by the high potential showed in laboratory investigations, the application of this method in real cases is rather difficult because of the increased complexity compared to that of the experimental setups.

That said, the choice of the most profitable leak detection technique is closely related to the characteristics of the examined case, especially in relation to the following criteria:

- *Applicability*, that is the existence of the conditions for a proper use of the technology;
- *Availability* in commerce of the necessary equipment;
- *Effectiveness*, in terms of accuracy and reliability of the returned results;
- *Sensitivity* of the instruments compared to the magnitude of the quantities that must be detected;
- *Adaptability* to local conditions determined by the hydraulic operation of the network or by environmental and climatic circumstances;
- *Economic balance* between the costs and the expected benefits.

#### **2.4.2. Pressure management**

The detection of the water leakages and their elimination through the repair or the replacement of the network pipes is the only way of ensuring the reduction of the non-revenue water. However, this activity is associated with very high costs and long operational times if applied at network scale, and is not consistent with the objective of improving the system efficiency as a whole.

In this regard, the most interesting perspectives are offered by the leakage control performed through the smart management of the water pressures in the network. Actually, this technique has shown to be very cost-effective especially in the mitigation of the background leakages, which represent the most difficult component of the water losses to be removed.

The basic concept of this methodology is that water networks are usually designed in order to ensure the minimum service requirements to customers in terms of water demand and pressure during all the day. Nevertheless, the strong variations of the consumption over time mean that the maximum system flow is achieved only for a few hours (peaks). During the remaining time, the network is run with water pressures that are largely greater than those that are strictly required for the proper operation. By virtue of the well-known relationship existing between flows and pressures, this situation is likely to increase the amount of water that is lost through the damages occurred to the pipes. For the same reason, it is expected that the reduction of the water pressures (in compliance with the system requirements) would result in the mitigation of the water losses throughout the whole network.

As highlighted by Thornton (2003), in the last few decades this proactive strategy has been recognized as “the essential foundation of effective leakage management”. Its success relies not only upon the reduction of the current level of water losses, but also on the decreased frequency in the occurrence of new leakages, which results in the extension of the working life of the water infrastructure.

On the other hand, it is important to acknowledge the major concerns that are related to the implementation of the pressure management in the real systems, which are namely: i) the potential changes in the water consumption (and, therefore, in the water utility revenue); ii) the effects on the network reliability in risk scenarios (e.g. fire fighting); iii) the hydraulic capacity of achieving the proper filling of the intermediate distribution storages (i.e. tanks); iv) the consequences for water quality.

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### **2.4.3. Leakage law**

In order to provide further insights into the application of this technique, a brief discussion about the pressure-leakage relationship (also known as the “leakage law”) is reported here. Several literature studies have been carried out on this topic, which have focused on both the structure of the leakage law and the calibration of its parameters.

Many authors have investigated the influence of the pipe material and of the shape and the size of the cracks on the lost discharge at varying the water pressure (Goodwin 1980; May 1994; Burnell and Race 2000; Farley and Trow 2003; Coetzer et al. 2006; Greyvenstein and van Zyl 2006; van Zyl and Clayton 2007; Ferrante et al. 2011; De Paola and Giugni 2010, 2012; De Paola et al. 2012). However, according to the Torricelli’s equation, the existence of an exponential relationship occurring between the pressure and the water leakage is commonly assumed. The most classical expression is the monomial formulation provided by Lambert (2001):

$$Q = \alpha P^\beta \tag{2.1}$$

where  $P$  is the water pressure and  $\alpha$  and  $\beta$  are the parameters of the leakage law, respectively named *discharge coefficient* and *leakage exponent*. In general, it has been observed that  $\alpha$  increases almost linearly with the area of the crack, and this confirms the experimental assumption about the structure of Eq. 2.1.

More uncertainties have been found in the estimation of  $\beta$ , which is not only influenced by the pipe material, but also by the shape of the crack (Greyvenstein and Van Zyl 2006). However, studies carried out on metallic pipes (i.e. ductile iron and steel) with circular and rectangular cracks have shown a substantial invariance of the leakage exponent with respect to the size of the leak. The detected values range between 0.47 and 0.50, and are consistent with the theoretical one provided by the Torricelli’s equation (De Paola and Giugni 2012).

In order to understand the effect of the pressure on the size of the leak, different models have been proposed, such as the FAVAD (Fixed and Variable Area Discharge) equation (May 1994). In other works, the pipe material behaviour has been addressed as well, in particular the viscoelastic deformation of the pipe walls in response to pressure variations (Ferrante 2012; van Zyl and Cassa 2014). These studies have shown the existence of a hysteresis in the deformation of the area of the leak against repeated cycles of increasing and decreasing pressure.

Only recently some authors have pointed out the problem of characterizing the real interaction of the leaking with the surrounding soil (van Zyl and Clayton 2007; De Paola et al. 2014), as this could affect the hydraulic boundary conditions for the outgoing flow. Actually, even though the pipes are usually placed under the ground level, in most of the studies experimental tests are performed on leaking pipes that are free to discharge water in the atmosphere. However, at present this topic has not been sufficiently investigated, and further research is required.

#### **2.4.4. Active Leakage Control through pressure management**

The reduction of the water pressures in the network can be effectively achieved through the introduction of Pressure Reducing Valves (PRVs). Such devices are able to perform the automatic regulation of the pressures by inducing minor head-losses that depend on their set points.

The operation of the PRVs can be either *fixed* or *time-modulated*. The first approach is obviously easier to implement, but limits the achievable benefits because of the particular attention that must be paid to meeting the minimum service requirements of customers. The time-modulated regulation is more complex and is performed by changing the set point of the valves during the day. In both cases, this adjustment requires a controlling device attached to the *pilot* of the PRV, that is the hydraulic circuit that rules the operation of the

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valve. The setting of the PRV can be changed at scheduled times or can be dynamically modulated according to the hydraulic response of the network (Fanner et al. 2007). In this case (also known as *flow-modulated* regulation) the acquisition of data in real time is required (e.g. through a Supervisory Control And Data Acquisition system, SCADA), as well as the possibility of performing the remote control of the devices.

The determination of the best number, location and setting of the PRVs to place in a water distribution network outlines an optimization problem, the solution to which has been addressed by many researchers to date. An extended overview of the different approaches that can be found in literature is presented in the Chapter 3. However, the implementation of the pressure management through the PRVs is facilitated by the preliminary *sectorization* of the network, which consists of its partitioning into sub-sectors that are usually named District Metering Areas (DMAs). Further details about these techniques are provided in the next section.

When pumping stations are present in the network, another interesting perspective for the optimal management of the water pressures is offered by the possibility of activating or de-activating the pumps depending upon the system demand. This methodology, usually referred as *pump scheduling*, can also produce significant savings in the consumed energy, especially if the efficiency of the working pumps is properly taken into account. These days this is made easier by the technological progress in the production of *power inverters*, which are also very useful for the limitation of the pressure surges that may be caused by unintended interruptions of the power supply (i.e. blackouts).

In recent years, another interesting methodology has been developed, namely the pressure management that is performed through the use of Pumps As Turbines (PATs). The basic idea of this innovative approach is the recovery of part of the mechanical energy that must be dissipated for reducing the water pressures through the production of electric energy (Chapallaz et al. 1992; Ramos and Borga 1999; Paish 2002; Isbasoiu et al. 2007; Giugni et al. 2009;

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Fontana et al. 2012). As pointed out by many authors, the adoption of pico-hydro schemes is not suitable for such application because of the difficulty in dealing with the daily and the seasonal patterns of the water demand and pressure, which dramatically modify the operation of the turbines. Moreover, the need for ensuring sufficient head for the proper service of the water system narrows the range of the usable turbines, which should be of the *reaction* type.

The above-mentioned reasons have led to an increasing attention towards the use of the PATs, which are conventional pumps installed for the reverse functioning. Among other qualities, these devices are characterized by a very large scale of production that makes their costs much lesser than those of turbines. This remarkable aspect is even more emphasized by the greater ease in finding spare parts. Nevertheless, the use of the PATs is affected by uncertainties that are due to the reduced availability of characteristic curves for the reverse operation. In addition, the performance of the PATs is generally worse than that of turbines because of the lack of a distributor that can optimize the impact of the water flow on the impeller blades.

The identification of the Best Efficiency Point (BEP) is therefore one of the most important issues to take into account in this application. The BEP corresponds to the maximum of the efficiency curve of the device, which is mainly obtained through experimental investigation and computational fluid dynamics (Gantar 1988; Williams 1995; Williams et al. 2003; Fernandez et al. 2004; Derakhshan and Nourbakhsh 2008; Carravetta et al. 2011). Another interesting method for assessing the performance of the PAT is the use of the *affinity law*, which makes it possible to extend the results obtained for a prototype to other devices having different impeller sizes and rotational speeds.

The improvement of the performance of the PATs can be achieved through the introduction of two PRVs, one in series and one in parallel. The former is intended to provide an additional dissipation when required, while the latter is used to bypass part of the flow. This approach is also known as Hydraulic Regulation (HR), whereas there is another method called Electric Regulation

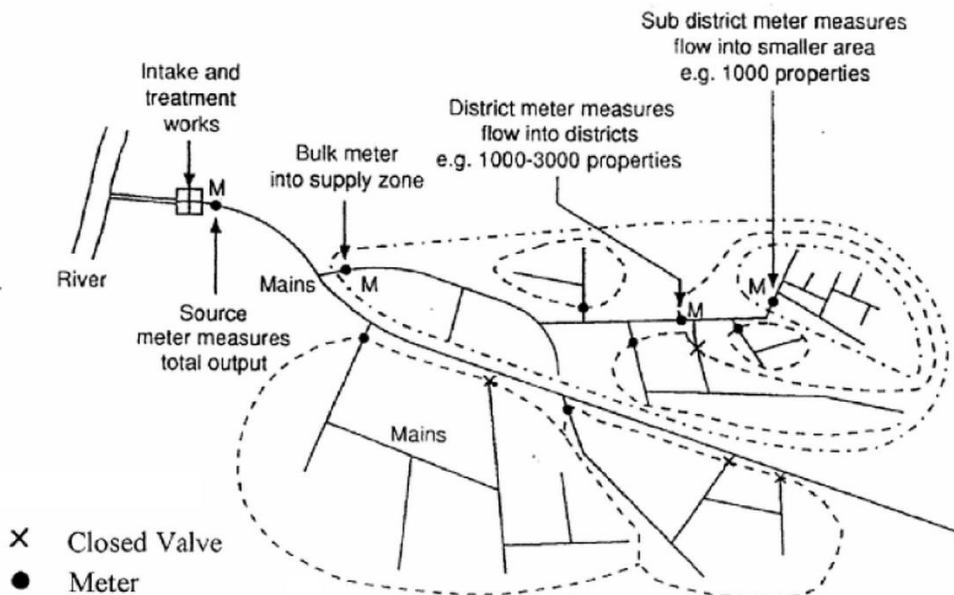
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(ER), which is based on the adoption of a power inverter for adapting the PAT operation (i.e. the rotational speed) to the hydraulic boundary conditions that are established in the network (Carravetta et al. 2012).

The use of the above-mentioned strategies for the optimization of the water distribution networks is a very challenging problem that has been largely discussed in the scientific literature. A more in-depth discussion of the optimization techniques that have been developed in this field is postponed to the next chapter.

### **2.4.5. Sectorization and district metering**

The *sectorization* of a water distribution network consists of the partitioning of the whole system into smaller and hydraulically discrete areas called District Metered Areas (DMAs). This technique was first introduced at the beginning of the '90s (Cheong 1993), and represents an alternative approach to the one that is typically adopted in designing water systems, which is primarily oriented to prefer highly meshed networks characterized by strong redundancy.



*Fig. 2.9 – Typical layout of a sectorized network (UKWIR 1999)*



This partitioning of the network is usually performed through the closure of network pipes by means of shut-off valves, while the supply for the DMAs is provided by a limited number of key mains, on which flow meters are installed. Generally speaking, the DMAs should not include the trunk mains, and the connection between different DMAs should be avoided. When this is not possible, flow-meters must be installed in order to measure the imports and the exports (Morrison et al. 2007). However, depending on the characteristics of the network (UK Water Industry Research Limited, UKWIR 1999; Fanner 2007), a DMA could be (see Fig. 2.9):

- supplied via single or multiple feeds;
- a discrete area (i.e. no flow into adjacent DMAs);
- an area which cascades into an adjacent DMA.

Among the general criteria concerning the district metering, the IWA guidelines (Morrison et al. 2007) indicate that the following factors should be taken into account in designing the DMAs (not in order):

- infrastructure condition and required economic level of leakage;
- size (geographical area and number of customer connections);
- types of customers (e.g. large metered customers should have their meters measured as export meters from the DMAs);
- housing type (i.e. blocks or flats or single family occupancy housing);
- variation in ground level;
- water quality considerations;
- pressure requirements and fire fighting capacity;
- number of valves to be closed and number of meters used to monitor flow (ideally minimized).

As highlighted in the same guidelines, the implementation of the designed sectorization in real networks should be carried out very carefully in order to check the hydraulic response of the system. Therefore, at least in a preliminary phase the boundary of the DMAs should not be considered definitive, because unexpected effects on the operating conditions could arise the need of providing adjustments. For the same reason, it is strongly suggested to delimitate the DMAs by closing pipes rather than cutting them.

The main goal of sectorization is that of providing an increased system control, which is primarily achieved through the continuous monitoring of the water flows in each area of the network. This also allows the water utilities to assess the current level of leakages in each sector and, consequently, to prioritize the leak detection activities. Moreover, as explained in Sec. 2.3.2, in partitioned networks the occurrence of new pipe bursts can be easily recognized through the analysis of the MNF.

The contribution offered by the establishment of DMAs to the smart management of the water pressures in the network is particularly interesting. In fact, the design of sectors can be carried out in order to obtain areas with similar hydraulic requirements, so that the network can be operated at the optimum level of pressure. This is also extremely useful for the Active Control of the water leakages, especially if the partitioning of the network is coupled with the introduction of PRVs.

However, it should be emphasized that sectorization itself can be considered as a pressure management technique. The closure of pipes reduces the number of loops in the network, and it may cause pressure drops that can be suitably exploited for reducing the amount of background leakages. This innovative approach is particularly addressed in this thesis, as explained in detail in Chapter 4, where a comprehensive discussion of the possible design criteria for the DMAs is presented as well.

Other significant benefits that can be achieved through the sectorization are those related to the water quality. In particular, it facilitates the collection of

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samples for the necessary physical-chemical analysis, but it also allows the water utility to isolate the areas eventually affected by contamination due to incidents or malicious attacks. In addition, it reduces the extent and the complexity of the mixing of different water sources (Herrera Fernandez 2011).

Nevertheless, there are some drawbacks that must be carefully taken into account in the establishment of DMAs. For example, especially in networks characterized by low operating pressures, the loss in the system resilience due to the reduced redundancy of the network layout can easily lead to the violation of the minimum service requirements. The same could happen in drinking water networks that are also responsible for the fire service. As for the quality, particular attention should be paid to the age of the water, which could be negatively affected by the reduced velocities and the more constrained paths that are allowed to the network flows.

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## **Optimization of water distribution networks: a literature review**

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The optimization of water distribution networks is a very challenging problem that has been addressed by many researchers in the last decades. These infrastructures can be considered as “complex systems” as they have heterogeneous composition, non-trivial configurations and multiple interconnected components (Yazdani and Jeffrey 2011). Such complexity is mainly created by their looped structures and by the governing laws of their physical behaviour, which are represented through both linear and non-linear equations. The significant uncertainties that characterize some of their components (e.g. the variability of the water demand) are also remarkable in this regard.

These issues significantly complicate the analysis of their performance and the planning tasks that must be carried out in their management, which mainly consist of design and operation problems. Even though these problems are obviously linked each other, they are usually solved separately in the analysis and in the technical management (Zhou and Simpson 2013).

To this aim, several approaches have been developed since the second half of the last century, which are based on the most successful optimization techniques that can be found in literature. A comprehensive overview of such methodologies is presented here, with particular reference to the optimization problems outlined in Chapter 2.

### **3.1. Optimal design of network pipes**

The design of water distribution networks basically consists of the assignment of the most appropriate diameters to the networks pipes. This selection should be performed with the aim of satisfying the water demand with sufficient pressure requirements (in different operating scenarios) while minimizing the cost.

As well as most of the problems concerning the water networks, the optimal design can be classified among the NP-hard problems (Yates et al. 1984) because it involves non-linear equations and discrete variables (i.e. the pipe diameters). Although the first developed approaches based on enumeration and complete trial-and-error techniques (Gessler 1985) are reliable global search methods, they were found to not be applicable to large networks because of the huge computational time required (Abebe and Solomatine 1998).

Therefore, the most traditional methods were based on the linearization and relaxation of the problem in order to facilitate the use of linear and non-linear programming (Alperovitz and Shamir 1977; Morgan and Goulter 1985; Kessler and Shamir 1989, 1991; Goulter et al. 1986; Fujiwara et al. 1987; Fujiwara and Kang 1990; Eiger et al. 1994). In such algorithms, the adoption of round-off functions was required to bring back the obtained values to discrete diameters. Moreover, these methods could not ensure the achievement of the global optimum and, sometimes, they caused infeasible solutions.

Significant progress in this field was registered with the introduction of meta-heuristic algorithms. Among these, Genetic Algorithms (GAs, Holland, 1975; Goldberg 1989) have shown very good performances in this kind of application (Simpson et al. 1994; Dandy et al. 1996; Savic and Walters 1997). These methods are based on the reproduction of the biological processes of survivability and adaptation that are observed in natural species, from which they inherited the adopted nomenclature (*population, chromosomes, genes, etc.*). Their basic concept consists of producing further improvements of the obtained

solutions through suitable combinations of the best ones that are found at each iteration.

The large success of GAs is mainly related to their effectiveness in the investigation of large search spaces with reduced time and use of computational resources. This is made possible by their structure, which works with “building blocks” (Goldberg 1989) and that facilitates their implementation and coding (also for parallel computing). Nevertheless, GAs are not able to ensure the global optimality of the provided solutions (Savic and Walters 1997). In fact, they require particular attention in the preservation of an adequate diversity in the evaluated population, which is crucial for avoiding the convergence towards local optima. This drawback is usually tackled by running several iterations of the optimization algorithms, which are carried out starting from different randomly generated solutions.

Other factors affecting the efficiency of the optimization are: i) the number of solutions to evaluate at each iteration (i.e. the *population size*); ii) the calibration of the parameters used for the recombination of the solutions; iii) the capacity of handling the problem constraints; iv) the selection of the stopping criteria. For example, the adoption of larger population size involves an increased number of evaluations of the objective function, and therefore, an enhanced capacity of finding better solutions.

A suitable selection of the parameters of the *genetic operators* (mainly *crossover* and *mutation*) can significantly improve the investigation of the objective space. In particular, the mutation operator should allow the algorithm to avoid local minima by preventing the population of chromosomes from becoming too similar to each other, thus slowing or stopping evolution. For this reason, in some approaches (Adaptive Genetic Algorithms, AGAs), the parameters of the genetic operators are even changed over the iterations of the algorithm in accordance with the values of the objective functions (Srinivas and Patnaik 1994).

As for the handling of the constraints, several approaches have been introduced in literature, that can be grouped into four main categories (Michalewicz and Schoenauer 1996):

- methods based on preserving the feasibility of the solutions;
- methods based on penalty functions;
- methods that able to discriminate between feasible and infeasible solutions;
- hybrid methods.

However, the most crucial issue is probably the selection of the stopping criteria, as this strongly influences the quality of the obtained solutions. The generational process is actually performed until a given condition is reached. Since the optimal solutions are not known a-priori, there is no information about the number of generations that should be carried out in order to ensure the convergence towards the global optimum. Even though there are not general rules for this selection, the terminating conditions can be suitably chosen from among the following:

- a solution is found that satisfies minimum criteria;
- a fixed number of generations or an allocated budget (computation time/money) is reached;
- the fitness of the best ranked solution does not improve after a given number of iterations;
- combinations of the above.

Despite the critical issues described so far, the use of the meta-heuristic approaches such as GAs for the design of pipe networks has gained increasing attention over time because of the very promising potential compared to traditional techniques (complete enumeration, linear and non-linear

programming). Simpson et al. (1994) proposed the application of a GA and the coding of the pipe sizes available for selection as binary strings. Their approach was improved by Dandy et al. (1996), who presented an improved GA based on a variable power scaling of the fitness function. This consisted of the use of a variable exponent, whose magnitude increased during the execution of the optimization algorithm. A new mutation operator (called *adjacency mutation*), and the *gray coding* (instead of the classical bitwise operator and the binary codes) were introduced as well. The procedure was successfully applied to the New York City Tunnels problem (Schaake and Lai 1969) showing better performance with respect to the previous approaches in both achieving feasible solutions and improving the value of the objective function.

Savic and Walters (1997) developed a computer model called GANET that implemented a genetic algorithm for the least-cost design of water distribution networks. The authors used three different examples to illustrate the potential of this model, namely the Two-loops (Alperovitz and Shamir 1977), the Hanoi (Fujiiwara and Khang 1990) and the New York Tunnels networks. The comparison with the previously published results showed that for a realistic range of parameters some of the latter did not satisfy the minimum pressure constraints, while this was efficiently achieved with the new optimization model.

In more recent years, several other meta-heuristic algorithms have been introduced in this field. Cunha and Sousa (1999; 2001) applied Simulated Annealing (SA) to the Hanoi and New York problems and they obtained very good results in terms of both solution quality (objective function value) and computing effort. Lippai et al. (1999) compared the performances of different commercial optimizers, thus highlighting the worst behaviour of the one based on the Tabu Search (TS, Glover and Laguna 1997) with respect to those of optimizers based on GAs. Other remarkable contributions are related to the use of: *ant colony* (Zecchin et al. 2005, 2006) and *particle swarm* optimization (Montalvo et al. 2008); *harmony search* (Geem et al. 2001; Geem 2006);

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*discrete state transition* algorithm (Zhou et al. 2012; Zhou and Simpson 2013); hybrid methods (Krapivka and Ostfeld 2009; Cisty 2010; Haghghi et al. 2011).

All of the cited works were aimed at the minimization of the total cost of the pipes. However, according to Todini (2000), this objective is not sufficient in ensuring the design of reliable water distribution networks, as it easily leads to solutions based on tree-shaped layouts. Such networks could have severe consequences in risk scenarios (e.g. pipe failures), which is the main reason for preferring looped and highly redundant structures. These issues were the basis for the formulation of the concept of *resilience*, which is a measure of the system reliability and that was used together with the total cost for the definition of multi-objective optimization problems for the design of water networks (Todini 2000; Prasad et al. 2003; Prasad and Park 2004; Farmani et al. 2005; Raad et al. 2009).

A comprehensive description of the optimization techniques that have been most successfully applied for the solution of these problems is reported in Wang et al. (2014). The authors compared the performances of several Multi-Objective Evolutionary Algorithms (MOEAs) on twelve benchmark problems having different “sizes” (*small, medium, intermediate* and *large* problems). The investigated algorithms included both referential MOEAs and hybrid approaches. In the former group, the authors considered: NSGA-II (Deb et al. 2002),  $\epsilon$ -MOEA (Deb et al. 2005) and  $\epsilon$ -NSGA-II (Kollat and Reed 2006; Tang et al. 2006). In the latter, the AMALGAM (Vrugt and Robinson 2007) and the Borg (Hadka and Reed 2013) algorithms were taken into account. As a result, NSGA-II confirmed itself as one of the most reliable MOEAs, and it was found to be particularly suitable for the two-objective optimization of water distribution networks. However, in order to ensure the achievement of reliable Pareto fronts when facing large sized problems, the use of different MOEAs should be considered as well.

### **3.2. Pump scheduling**

The operation of the pumping stations in water distribution network is responsible for a significant amount of electric energy consumption, and involves very high costs for the water utilities whilst having environmental consequences in both terms of over-exploitation of natural resources and emission of greenhouse gases.

Therefore, particular attention is required in the improvement of the efficiency of these facilities, which can actually be achieved through the optimal control of their operation. Moreover, as highlighted in Chapter II, the smart management of the pumping stations can be also related to significant savings in the non-revenue water, as it influences the amount of the real water losses in the networks.

The optimization of the pump-scheduling has been studied by many authors with different approaches. The first methods that were developed consisted of linear (Jowitt and Germanopoulos 1992), non-linear (Chase and Ormsbee 1993; Yu et al. 1994), dynamic (Lansey and Awumah 1994), mixed, and integer programming techniques that mainly aimed at the optimization of a single objective, which was the cost due to energy consumption. Nevertheless, as pointed out by some authors (Lansey and Awumah 1994), the maintenance cost of the pumps should be taken into account as well. Given the difficulties in estimating this cost, and provided that the cycle life of a pump is affected by the number of switches, this is often adopted as surrogate objective to be minimized.

An overview of the classical approaches can be found in Ormsbee and Lansey (1994), who reported several methods classified according to: i) the hydraulic model, which could be based on mass balance relationships, regression curves, simplified hydraulics or full hydraulic simulation; ii) the demand model, in terms of both estimation and allocation; iii) the optimization model, which was characterized on the basis of the model formulation and the

control algorithm. The most straightforward methods were those assuming explicit decision variables, namely the fraction of time intervals during which each pump was operating.

However, as for the optimal design of pipes, also in this field the meta-heuristic approaches (Mackle et al. 1995; Savic et al. 1997; van Zyl et al. 2004; López-Ibáñez et al. 2008) have proven to be robust alternatives because of the reduced need for simplifications and basic assumptions compared to traditional strategies. On the other hand, from a practical point of view, the basic population-based techniques still suffer from the fundamental drawback of requiring a high number of evaluations to achieve convergence. For this reason, hybrid optimization models (Giacomello et al. 2013) have been introduced more recently, with the particular aim of facilitating the real-time operational control especially in complex water networks with large number of pumps.

### **3.3. Pressure management through PRVs and PATs**

This is probably the most debated problem concerning the optimization of water systems. In particular, in early researches the optimal setting of PRVs located at known positions in the networks was addressed. One of the first examples was that proposed by Sterling and Bargiela (1984), who developed an optimization method based on the Successive Linear Programming (SLP). This method consisted in the iterative linearization of the equations in order to solve the problem with the linear programming. The sum of the water pressures at network nodes was adopted as the objective function to be minimized, while only the governing equations were considered as problem constraints. A similar approach can be found in Germanopoulos and Jowitt (1989), who also introduced the terms related to the water losses among the constraints. Nevertheless, the linearization procedure used in this case was only based on the pressures of the previous iteration, whereas it did not include the valve settings.



Jowitt and Xu (1990) proposed a different formulation of the problem, which was based on the minimization of the volume of the water leakages instead of the total pressures in the network. The linearization procedure was improved as well by taking into account the settings of the valves of the previous iterations. However, one of the most successful definitions of the objective function was introduced by Vairavamoorthy and Lumbers (1998), who considered the following:

$$Z = \sum_{i=1}^N (P_i - P_i^*)^2 = \min! \quad (3.1)$$

In other words, in agreement with the theory relevant to the pressure-leakage relationship, the authors matched the mitigation of the water losses with the minimization of the sum of the squared differences between the actual ( $P_i$ ) and the target ( $P_i^*$ ) pressures (i.e. the service requirements) for each node in the network. An approximate method based on the solution of a sequence of quadratic programming sub-problems (SQP) was adopted in this study, which showed better convergence properties compared to those applied in previous works. The same authors also pointed out that, in order to maximize the effectiveness of the pressure management, the optimal location of the PRVs should be taken into account as well.

This problem has been mainly addressed with the use of meta-heuristic algorithms. Savic and Walters (1995, 1996) developed models based on steady-state hydraulic simulations for the location of valves in water distribution networks. However, in these cases, the authors only considered valves in the complete open or closed position. Reis et al. (1997) embedded a linear programming algorithm in a GA for the determination of the optimal valve settings.

Araujo et al. (2006) developed a two-stage method based on the use of GAs for solving both the problems of location and setting of the PRVs. In the first

step, the optimization of the number and positions of the valves was performed through the assignment of an additional roughness to candidate pipes. Then, the adjustment of the valve openings (i.e. the settings) was carried out in order to optimize the pressures in the network. More recently, a single level optimization involving a GA was adopted by Ali (2014).

A different approach was proposed by Liberatore and Sechi (2009), who used a meta-heuristic approach called Pressure Reference Method (PRM). In this technique, a reference pressure value and hydraulic simulations were used to extract from the network layout the possible candidate pipes for the location of the PRVs. These were the pipes for which the end nodes were one above and the other below the reference value. Then, in a subsequent step, the status of the valve (*active, open or closed*) was defined. The Scatter-Search meta-heuristic algorithm (Glover 1999) was implemented as well. Furthermore, the authors introduced a powerful scheme for modelling the valve insertion on a pipe, which was very useful for avoiding the unintended isolation of network areas that could be due to the working direction of the PRVs.

In the context of multi-objective optimization, Nicolini and Zovatto (2009) and Nicolini et al. (2011) investigated the problem of minimizing both the number of valves and the water leakages in the network, while Creaco and Pezzinga (2014) developed a comprehensive approach for the attenuation of the water leakages through the pipe replacement and the use of control valves. The real-time control of the PRVs has been addressed as well (Campisano et al. 2010, 2012; Creaco and Franchini 2013), which consists of the dynamic adjustment of the valve setting for attaining the desired pressure heads at specific nodes in the network.

Finally, in recent years some researchers have addressed the optimization of the water pressures in the network through the use of PATs, with the double aim of achieving the mitigation of the water leakages and the production of electric energy as well. Fontana et al. (2012) applied a GA-based model to a real network in order to assess the optimal location of PRVs for reducing the

leakages. Then, they replaced some of the valves with PATs for hydropower generation and they estimated the potential revenues due to the energy recovery. However, the authors emphasized that the optimal location of the PRVs does not ensure the maximization of the energy production, and that a specific objective function should be adopted for this purpose.

In this regard, an explicit formulation can be found in Tricarico et al. (2013), who developed a multi-objective optimization approach taking into account the minimization of the pump cost and of the surplus pressure, as well as the maximization of annual income due to the operation of the PATs.

### **3.4. Network segmentation / sectorization**

The partitioning of a water distribution network into suitable DMAs (also referred as *network segmentation* or *sectorization*) is not a trivial issue, and its definition can be addressed with several approaches and criteria. Many researchers have investigated this problem with different goals and optimization techniques.

Tzatchkov et al. (2006) presented an algorithm based on graph theory for obtaining the number of independent supply sectors in a network layout through the *Last-In-First-Out (LIFO) stack approach*.

Sempewo et al. (2009) developed a spatial analysis zoning approach based on the METIS graph partitioning tool (Karypis and Kumar 1995). The proposed technique followed the analogy with the distributed computing methodology of equally allocating workloads among processors. The procedure aimed at the division of the network in DMAs with similar sizes while minimizing the number of links to be closed (also called *edge cut*). However, as stated by the same authors, although the method was effective in the demarcation of contiguous districts, the quality of the provided solution degraded when multi-objective partitioning was taken into account. Moreover, significant uncertainties were produced when the number of DMAs was increased.

Di Nardo and Di Natale (2011) developed a methodology based on the Dijkstra's shortest path algorithm (Dijkstra 1959). The optimal partitioning of the network was obtained through the use of energy indices and the computation of the paths with the least hydraulic resistance from the sources to every demand node in the network.

Gomes et al. (2012a) adopted a similar approach, also introducing user-defined criteria and the Simulated Annealing algorithm in order to identify the location of the boundary valves and the necessary pipe reinforcement/replacement for meeting the velocity and the pressure requirements. Subsequently, the same technique was improved with the implementation of pressure management (Gomes et al. 2012b). However, both the developed methodologies were not completely automated, and a significant contribution by the operator was required in the optimization.

Herrera et al. (2012) proposed a semi-supervised method named *multi-agent adaptive boost clustering*. This complex technique considered both the network features (e.g., node elevations and demands) and economic issues (i.e. the edge cut) for the sectorization. Nevertheless, it was only applicable to cases in which the number of DMAs was lower than the number of source nodes.

Alvisi and Franchini (2013) presented an automatic procedure implementing the Breadth-First Search algorithm (Moore 1959). In this case the DMAs were created by grouping the network nodes to the selected "sources" according to the distance and the cumulative water demand. The different designs of the network sectorization were compared in order to find the most resilient one.

A different approach based on the *community structures* was investigated by Diao et al. (2013). In this technique, the DMAs were identified with the "communities", which were defined as groups of nodes characterized by a high density of links between them. The developed algorithm started by assuming each node as a single community, and then continued by joining the communities together until a pre-fixed upper limit for the size of the DMAs was

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reached. In other works, the use of the *modularity index* has been addressed (Scibetta et al. 2013; Giustolisi 2014). However, these approaches were mainly focused on the system topology, while they addressed with less emphasis the hydraulic performance of the partitioned network and the specific purposes of the sectorization that were discussed in Chapter 2.

More technical principles were adopted in the methodology by Di Nardo et al. (2014a), in which the isolation of the DMAs and the assignment of a specific source to each of them were assumed as primary purpose. To this aim, the authors used a combination of graph theory principles and heuristics for the minimization of the amount of dissipated power in the water network. The protection of the water against malicious attacks was investigated in Di Nardo et al. (2014b), which used the EPANET (Rossman 2000) routines for the simulation of the water quality in risk scenarios.

Another interesting approach was proposed by Ferrari et al. (2014), who took the optimization of the size of the DMAs, the connectedness of each district to a water source and the absence of links between different DMAs as design criteria. A recursive procedure was adopted for the design of the network partitioning, which was subsequently verified in order to assess the reachability of each DMA from the source nodes.

Despite the number of contributions, several criticalities can be found in the approaches developed so far. As highlighted by Hajebi et al. (2014), the proposed methods mainly suffer from one or more of the following limitations: i) the applicability only to small networks; ii) the neglected inclusion of the ground elevations among the design criteria; iii) the lack of objective functions related to the hydraulics of the network. The same authors introduced a multi-objective approach based on the use of NSGA-II (Deb et al. 2002). Nevertheless, the large number of considered objectives significantly reduced the readability of the obtained results.

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# Methodology

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In this chapter a new methodology for the automatic and optimal partitioning of a WDN into DMAs is presented and discussed. The proposed approach is based on the review of the possible design criteria, both retrieved from the literature and the technical best practice. In order to provide the best solutions under a multi-objective perspective, a large set of them is taken into account for the development of the partitioning strategy.

The mathematical formulation of the optimization problem is also provided here. The objective functions are defined with the aim of providing clear information to the decision makers and for improving the readability of the results. The problem constraints are introduced as well.

Given the usually large size of the real networks and the non-linearity of the involved equations, it would be very difficult to find the global optimum of the problem through analytical derivation. In the proposed approach, this issue is tackled through the use of an evolutionary algorithm and applications from the graph theory.

The optimization algorithm and all the adopted procedures and tools are described and detailed through the use of schematics and references to a simple test case. The structure of the algorithm is also designed with the aim of allowing an easy implementation with a software, that in this thesis is performed through the Matlab programming language.

As a result, a number of different and hydraulically feasible optimal designs of DMAs can be automatically obtained starting from the hydraulic model of the WDN, once the input parameters and the analysis options are specified.

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#### **4.1. Design criteria**

Different design criteria for sizing the DMAs are taken into account in this work. In particular, they belong to three main fields:

- network topology;
- financial issues;
- network hydraulics.

First of all, the topological features of the WDN are addressed. Following the assumptions from the classical approach (Morrison et al. 2007), the DMAs should supply a limited number of customers, commonly assumed between 500 and 5,000. This best practice is intended for the establishment of reasonably sized DMAs and for the limitation of the differences between their extent.

Furthermore, DMAs should have a limited number of connections with the main trunks (or with other DMAs). This feature reduces the number of flow meters required for monitoring the input and output flows, and it allows a finer control on the system functioning.

In addition, the ground elevations should be taken into account, and in particular the differences in the altitude of the network nodes within the same DMA should be minimized. This criteria, that is usually neglected in most of the literature studies (Hajebi et al. 2014), is useful for identifying a unique pressure target level against which it is possible to reduce the leakages without violating the minimum pressure requirement at demand nodes.

The cost of the DMA creation project must be considered as well. Most of the methodologies proposed so far only consider the cost of the construction cost (i.e. the cost of the shut-off valves and of the flow meters required for setting the boundaries of the DMAs). This cost can be easily evaluated according to the diameters of the pipes on which the devices have to be installed. However, the approach presented here aims to deal with the total operative cost for the partitioned WDN, which also includes the cost incurred

from water leakages. The evaluation of the latter requires a proper localization of the leakages into the network and the characterization of the pressure – leakage relationships.

The evaluation of the unit cost of the lost water can be carried out with several techniques. However, this estimate must surely take into account the operational cost due to the “production” of drinking water (abstraction, treatment, transport, storage, delivery). Furthermore, since water is a natural resource, the marginal cost due to the externalities generated by the loss of large volumes of water should be considered as well (Tripartite Group 2002).

Finally, the hydraulic functioning of the network is addressed. The compliance with the hydraulic constraints must be verified (mass balance equations, head-loss formulas, pump curves, etc.), as well as the minimum service requirements at nodes (in terms of both water demand and pressure).

As pointed out previously, the closure of network pipes for the establishment of the DMAs can significantly reduce the redundancy of the WDN layout. This may lead to an excessive loss in the system resilience against unintended disconnections caused by pipe failures or significant changes in the spatial or temporal distribution of the water demands (e.g. fire service). Therefore, particular attention should be paid to the preservation of the hydraulic reliability of the WDN, even though this aspect is in contrast with the reduction of the water pressures for the mitigation of the leakages.

## **4.2. Problem formulation**

### **4.2.1. Objective functions**

The entire set of design criteria described in the previous section is taken into account for the development of the optimization algorithm. However, from a mathematical point of view, the formulation of the problem follows the approach by Gomez et al. (2013), where a total cost function is compared to the

change in the hydraulic performance of the WDN in terms of reduction of the network resilience.

This selection focuses the optimization on the two main aspects involved in the design of the network segmentation. Nevertheless, the other criteria are properly considered in the intermediate steps of the algorithm execution, as explained in details in the next sections.

The Total Cost Function (*TCF*) is defined as the total cost evaluated on a single year of operation, and it can be calculated as follows:

$$TCF = w_B (r \cdot C_B) + w_L (365 \cdot C_L) = \min! \quad (4.1)$$

where  $C_B$  is the construction cost,  $C_L$  is the daily cost due to the water leakages,  $r$  is an annual discounting rate and  $w_B$  and  $w_L$  are two weights whose definitions are provided later in this section.

Considering the total number of pipes intercepted with shut-off valves ( $N_{SV}$ ), the number of accesses to the DMAs (i.e. the number of flow meters,  $N_{FM}$ ) and the respectively unit costs ( $C_{SV}$ ,  $C_{FM}$ ), the required investment is calculated as follows:

$$C_B = \sum_{j=1}^{N_{SV}} C_{SV,j} + \sum_{k=1}^{N_{FM}} C_{FM,k} \quad ; \quad j = 1, \dots, N_{SV} \quad ; \quad k = 1, \dots, N_{FM} \quad (4.2)$$

The cost due to the water leakages can be evaluated with reference to the pressure – leakage relationship in the monomial formulation (Lambert 2001):

$$Q_i = \alpha_i P_i^{\beta_i} \quad ; \quad i = 1, \dots, N_{leak} \quad (4.3)$$

where  $\alpha_i$  and  $\beta_i$  are respectively the discharge coefficient and the emitter exponent at the  $i$ -th leaking node, while  $P_i$  is the corresponding value of the water pressure.



Assuming that the hydraulic simulation is performed with reference to a daily time pattern for the water demand (consisting of  $T$  time steps), the value of  $C_L$  is calculated through the following:

$$C_L = C_{WL} \cdot \sum_{i=1}^{N_{leak}} \sum_{t=1}^T (\alpha_i P_{i,t}^{\beta_i} \Delta t) \quad ; \quad i = 1, \dots, N_{leak} \quad ; \quad t = 1, \dots, T \quad (4.4)$$

in which  $C_{WL}$  is the unit cost of lost water,  $t$  is the generic time step and  $P_{i,t}$  is the pressure at the  $i$ -th leaking node (assumed constant during the whole duration of the time step  $\Delta t$ ).

The provided definition of the  $TCF$  is intended to give to the decision maker the possibility of introducing his preferences in the optimization. By changing the values of the two weights  $w_B$  and  $w_L$  in Eq. 4.1, it is possible to put the desired emphasis on the two different cost items considered in this approach. For example, one can be interested in a long term payback that is mainly achievable through the reduction of the water leakages. Conversely, in case of a very limited available budget, the decision-maker would more probably focus on minimizing the construction cost. The weights should be selected according to the following:

$$w_B, w_L \in [0;1] \quad ; \quad w_B + w_L = 1 \quad (4.5)$$

It must be highlighted that the presence of the weights into the Eq. 4.1 modifies the dimension of the  $TCF$ , which cannot be expressed in terms of currency. Moreover, the selection of the weights is arbitrary and its influence on the results of the optimization should be carefully taken into account (and eventually evaluated through a sensitivity analysis). The same can be said for the evaluation of the annual discounting rate.

However, it is worthwhile noting that the provided formulation does not cause any loss of generality. The particular case in which equal weights (0.50

each) are adopted involves the evaluation of a *TCF* that is half the value of the annual total operative cost expressed in the adopted currency.

Against the *TCF*, the reduction of the network reliability due to the closure of network pipes is evaluated. Several performance indicators can be found in literature for this purpose. A definition of resilience can be found in Todini (2000), who proposed the following Resilience Index:

$$I_r = 1 - \frac{P_D}{P_{D,max}} \quad (4.6)$$

in which  $P_D$  is the dissipated (or internal) power, while  $P_{D,max}$  is its maximum value corresponding to the maximum power that would be dissipated in the network pipes in order to satisfy the constraints in terms of water demand and head at the nodes.

The dissipated power can be obtained by subtracting the power delivered to users in terms of flow and head at each demand node (external power,  $P_N$ ) from the total power available at the source nodes ( $P_A$ ):

$$\begin{aligned} P_A &= \gamma \sum_{s=1}^{N_s} Q_s H_s & ; & \quad s = 1, \dots, N_s \\ P_N &= \gamma \sum_{i=1}^{N_n} Q_i H_i & ; & \quad i = 1, \dots, N_n \\ P_D &= \gamma \sum_{l=1}^{N_l} Q_l \Delta H_l = P_A - P_N & ; & \quad l = 1, \dots, N_l \end{aligned} \quad (4.7)$$

where  $\gamma$  is the specific gravity of water;  $N_s$ ,  $N_n$ ,  $N_l$  are the numbers of source nodes, of demand nodes and of network pipes, respectively;  $Q_s$  and  $H_s$  are the discharge and the total head relevant to the  $s$ -th source;  $Q_i$  is the water demand delivered at the  $i$ -th demand node with total head  $H_i$ ;  $\Delta H_l$  is the head-loss caused by the discharge  $Q_l$  through the  $l$ -th pipe.

If  $\hat{H}_i$  is the minimum head required at the  $i$ -th demand node, the maximum value of the dissipated power is given by:

$$P_{D,\max} = P_A - P_{N,\min} = \gamma \sum_{s=1}^{N_s} Q_s H_s - \gamma \sum_{i=1}^{N_n} Q_i \hat{H}_i \quad (4.8)$$

Consequently, the Resilience Index can be expressed as follows:

$$I_r = 1 - \frac{P_D}{P_{D,\max}} = 1 - \frac{P_A - P_N}{P_A - P_{N,\min}} = \frac{P_N - P_{N,\min}}{P_A - P_{N,\min}} = \frac{\sum_{i=1}^{N_n} Q_i (H_i - \hat{H}_i)}{\sum_{s=1}^{N_s} Q_s H_s - \sum_{i=1}^{N_n} Q_i \hat{H}_i} \quad (4.9)$$

The Resilience Index provides clear information about the hydraulic performance of the WDN: high values of  $I_r$  are associated to low dissipated power and, therefore, to increased network reliability.

In order to assess more directly the effects of the sectorization on this feature, a particular derivation of the Resilience Index is adopted here, namely the Resilience Deviation Index ( $I_{RD}$ ) proposed by Di Nardo and Di Natale (2011):

$$I_{RD} = 1 - \frac{I_{r,d}}{I_{r,nd}} \quad (4.10)$$

The Resilience Deviation Index operates a comparison between the values of the Resilience Indices of the WDN before ( $I_{r,nd}$ ) and after the establishment of the DMAs ( $I_{r,d}$ ). A different expression can be obtained with a little manipulation:

$$I_{RD} = 1 - \frac{I_{r,d}}{I_{r,nd}} = 1 - \frac{\frac{\sum_{i=1}^{N_n} Q_i (H_i - \hat{H}_i)}{P_A - P_{N,\min}}}{\frac{\sum_{i=1}^{N_n} Q_i (H_{i,nd} - \hat{H}_i)}{P_A - P_{N,\min}}} = \frac{\sum_{i=1}^{N_n} Q_i (H_{i,nd} - H_i)}{\sum_{i=1}^{N_n} Q_i (H_{i,nd} - \hat{H}_i)} \quad (4.11)$$

where  $H_{i,nd}$  and  $H_i$  are the total heads retrieved from the hydraulic simulation at the  $i$ -th demand node in the original and in the partitioned configuration of the WDN, respectively.

Since the closure of pipes causes a reduction in the redundancy of the system layout, the reliability of the network is expected to be decrease after the sectorization. Consequently, the values of  $I_{RD}$  are expected to range between 0 and 1 and to increase with the decrease of the hydraulic performance.

Therefore, the preservation of the WDN resilience can be achieved by minimizing the maximum value of  $I_{RD}$  over time. The final expression of the corresponding objective function is given by the following:

$$I_{RD,max} = \max_t \{I_{RD}\} = \min! \quad ; \quad t \in \{1, \dots, T\} \quad (4.12)$$

#### **4.2.2. Constraints**

Among the problem constraints, first of all the hydraulic feasibility of the provided solutions is considered. In particular, the discharge in the WDN pipes and the head at nodes must satisfy the hydraulics of the system, namely the following:

- mass balance equations;
- head-loss formulas;
- minimum and maximum levels in tanks;
- pump curves.

In this methodology, the solutions to these equations are all calculated by the hydraulic simulator, whose details are discussed later in the text.

Furthermore, the minimum service requirements for customers must be verified in the partitioned WDN. Assuming  $P_{min}$  as the minimum pressure target

that must be achieved in order to supply the requested water demand at nodes, the following conditions must be satisfied:

$$P_{i,t} \geq P_{\min} \quad \forall i \in \{1, \dots, N_n\}, \forall t \in \{1, \dots, T\} \quad (4.13)$$

A financial constraint can also be defined in case of limited budget for the construction cost ( $C_{B,max}$ ). The corresponding function can be easily formulated as follows:

$$C_B \leq C_{B,max} \quad (4.14)$$

Finally, the design criterion related to the number of customer connections served by each DMA is addressed. As stated before, this number should range between 500 (*MinCon*) and 5,000 (*MaxCon*). However, in most cases the information about the number of customers served by each node in the hydraulic model of the network is not properly detailed. Conversely, the demand served by each node is an unavoidable input parameter for running the simulations.

For this reason, in the proposed approach this criterion can be formulated in terms of water demand served by each DMA. The lower and the upper bounds for this quantity are calculated as follows (Ferrari et al. 2014):

$$\begin{aligned} Q_{d,\min} &= \frac{MinCon}{TotCon} \cdot Q_{tot} \\ Q_{d,\max} &= \frac{MaxCon}{TotCon} \cdot Q_{tot} \end{aligned} \quad (4.15)$$

and therefore, the corresponding constraint could be considered:

$$Q_{d,\min} \leq Q_d \leq Q_{d,\max} \quad (4.16)$$

where  $Q_d$  is the sum of the base demands at demand nodes within the  $d$ -th DMA,  $Q_{tot}$  is the sum of the base demands of all the network nodes and  $TotCon$  is the total number of customer connections.

However, given the purely informative nature of the best practice limits, in the proposed approach just a corresponding “fuzzy” constraint is defined (Fig. 4.1). In brief, when the values of  $Q_d$  for all the DMAs are in the above mentioned range, the solutions are considered feasible. Otherwise, if at least one  $Q_d$  falls outside the range, but within the 50% of the bounds, the solutions are accepted with a probability ( $\lambda_d$ ) that linearly decreases with the distance from the limits. In all the other cases, the solutions are considered infeasible.

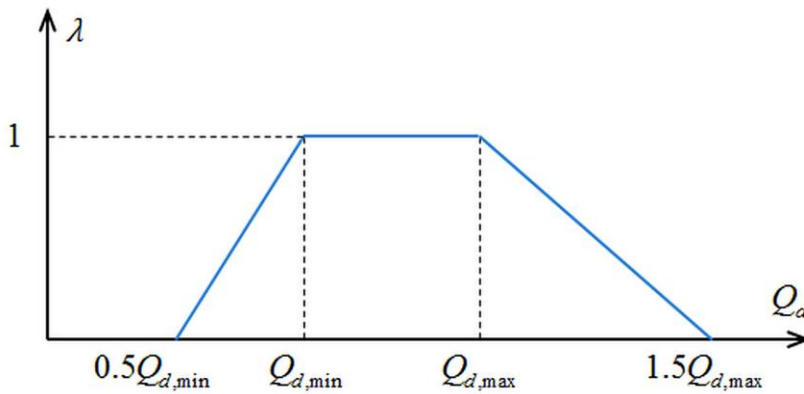


Fig 4.1 – Fuzzy constraint for water demand in DMAs

The mathematical formulation of  $\lambda_d$  is reported below:

$$\lambda_d = \begin{cases} 0 & \text{if } Q_d \leq 0.5Q_{d,\min} \\ (Q_d - 0.5Q_{d,\min}) / 0.5Q_{d,\min} & \text{if } 0.5Q_{d,\min} < Q_d < Q_{d,\min} \\ 1 & \text{if } Q_{d,\min} \leq Q_d \leq Q_{d,\max} \\ (1.5Q_{d,\max} - Q_d) / 0.5Q_{d,\max} & \text{if } Q_{d,\max} < Q_d < 1.5Q_{d,\max} \\ 0 & \text{if } Q_d \geq 1.5Q_{d,\max} \end{cases} \quad (4.17)$$

Therefore, given a randomly generated number  $\theta$  having values in the interval  $[0;1]$  and assuming that  $NDMA$  is the number of DMAs in the current solution, the constraint related to the size of the DMAs is reduced to the following:

$$\lambda_d \geq \theta \quad ; \quad \forall d \in [1; NDMA] \quad (4.18)$$

### 4.3. Optimization algorithm

According to the description provided in the previous section, a bi-objective optimization problem is formulated. It consists of the minimization of the two objective functions reported in Eqs. 4.1 and 4.12, subject to the constraints in Eqs. 4.13, 4.14 and 4.18 (in addition to the verification of the governing equations that is checked by the hydraulic simulator).

The optimization algorithm outlined in the flowchart of Fig. 4.2 provides the solution to this problem. The process starts with an initialization phase, in which the assignment of the input parameters is checked and preliminary calculations are performed. Then, the input parameters are passed to NSGA-II, which is the multi-objective evolutionary algorithm adopted in this work.

The NSGA-II algorithm starts with the generation of a desired number of possible solutions to the optimization problem. Each solution consists of a string of values, one for every decision variable, that are randomly selected between the related lower and upper bounds.

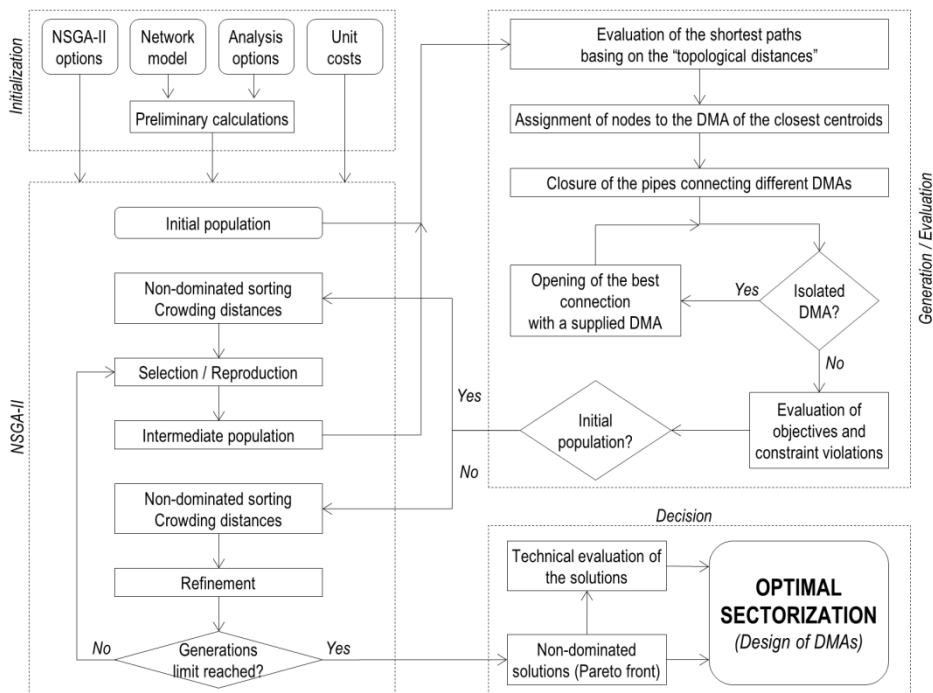


Fig. 4.2 – Schematic of the optimization algorithm

The set of decision variables includes the following:

- the number of DMAs ( $NDMA$ ) in the current solution (integer, ranged between 1 and  $NDMA_{max}$ );
- the node indices for a number of DMA centroids equal to  $NDMA$  (integer, ranged between 1 and  $N_{tot}$ );
- a blending factor ( $\varphi$ ) for the evaluation of a “topological distance” between nodes (real, ranged between 0 and 1).

It is important to highlight here one of the most innovative feature of the proposed methodology. The number of DMAs is often considered as an input parameter for the analysis. Conversely, in this case only its upper bound ( $NDMA_{max}$ ) should be assigned, while the optimal number of DMAs is obtained as a result of the optimization. Therefore, less information is required and the optimization is not biased by preliminary assumptions.

Another point of interest is that the set of decision variables includes both integer and real values. Particular attention should be paid in the assignment of the parameters of the genetic operators (crossover, mutation) in order to ensure an efficient exploration of the objective space. Further details are provided in Sec. 4.3.4.

The “centroids” could be intended as the barycentric nodes of the DMAs. Their indices are selected among those of all the network nodes, whose number is  $N_{tot}$ . At each generation of the optimization algorithm, the DMAs are built starting from the centroids, as explained later in the text. However, since the number of DMAs is assumed as an independent variable, only the first  $NDMA$  centroids are actually taken into account for generating the corresponding solution. This procedure is completely described in Section 4.3.2, as well as the use of the blending factor  $\varphi$ . However, a preliminary introduction of the latter is provided in the next section.



Once the solutions are created, it is possible to evaluate the objective functions and the constraints. Then, the best solutions are selected and combined through the genetic operators for creating a new population.

The operations mentioned above are repeated until the desired number of generations is reached. The obtained set of non-dominated solutions (i.e. the Pareto front) represents the solution to optimization problem and provides support to the decision making. In addition, a set of relevant quantities is extrapolated in order to facilitate the technical evaluation of the obtained projects of network segmentation.

### **4.3.1. Initialization**

First of all, in this step the general information about the WDN must be provided. This information consists of the following:

- network layout (i.e. the network geometry);
- total number of customer connections;
- minimum service pressure at demand nodes.

In case the total number of customer connections (*TotCon*) is not directly available, its value can be estimated according to the average daily demand supplied at nodes (i.e. the *base demand*) and assuming a realistic value for the average per-customer daily consumption (e.g. in litres/customer/day).

The minimum service pressure at nodes is the minimum pressure that is required for ensuring proper supply of water at customers. Even though this value depends on many factors (height of buildings, type of customers, service standards, etc.), in most cases a unique value could be identified for the whole network (e.g. the maximum).

Furthermore, other relevant features are directly gathered from the hydraulic model of the WDN, in order to obtain information about the following components:

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- nodes (demand nodes, pipe junctions);
- links (pipes, valves, pumps);
- water sources (reservoir, tanks);
- time patterns (water demand, pump schedules, etc.);
- operational rules and controls;
- total duration of the hydraulic simulation and number of time steps.

In particular, a detailed list of the relevant parameters for each hydraulic component is reported in Tab. 4.1.

The nodes are classified into *junctions* (nodes connecting different pipes) and *demand nodes*, which are those that deliver the water demand to customers. However, both categories can be also be modelled as *emitters* for reproducing pressure-dependent water leakages. In this case, the corresponding parameters (discharge coefficient and emitter exponent) must be provided.

The water sources are divided into two groups: the *reservoirs*, which are assumed to have by an infinite capacity and a fixed total head, and the *tanks*, which are characterized by a limited volume and a variable water level. The minimum, the maximum and the initial level (i.e. at the beginning of the hydraulic simulation) for all the tanks must be specified, as well as the corresponding water volumes (or a characteristic dimension to use for their evaluation).

❖ Junctions	+ node index + ground elevation + discharge coefficient + emitter exponent
❖ Demand nodes	+ index + ground elevation + base demand + demand pattern + discharge coefficient + emitter exponent
❖ Reservoirs	+ node index + total head

❖ Tanks	+ node index + elevation + base area + minimum level + maximum level + initial level
❖ Pipes	+ link index + start node + end node + length + diameter + roughness + initial status
❖ Pumps	+ link index + start node + end node + pump curve
❖ Valves	+ link index + start node + end node + diameters + type + setting

*Tab. 4.1 – Network components and parameters required for the optimization*

In the hydraulic modelling of WDNs, the *valves* and the *pumps* are usually modelled as links. For this reason, similar information to that required for *pipes* must be provided (e.g. the start node and the end node). However, different parameters must be defined to characterize their hydraulic behaviour. For example, the setting parameter for each valve is required, which depends on the valve type (e.g. flow control valve).

This characterization of the network allows a preliminary hydraulic simulation, from which other useful quantities are detected. These are: i) the flows in pipes; ii) the total heads at demand nodes; iii) the amount of water leakages in the starting configuration (i.e. the non-partitioned network).

In addition to the aforementioned features, the analysis options must be provided in this phase, in order to carry out other preliminary calculations. In

particular, the following must be specified: the upper bound for the number of DMAs ( $NDMA_{max}$ ); the minimum ( $MinCon$ ) and the maximum ( $MaxCon$ ) numbers of customer connections per DMA; the budget limit ( $C_{B,max}$ ) for the construction cost (in currency); the annual discounting rate (in percentage); the weights to adopt in the Total Cost Function ( $w_B$  and  $w_L$ ).

Then, the lower and the upper bounds for the “size” of the DMAs (i.e. the total demand served by each DMA, see Section 4.2.2) are calculated. Furthermore, two  $n \times n$  matrices are built (with  $n$  being the number of nodes in the network graph, including water sources), which consist of the weights on the network edges (i.e. the links connecting different nodes) evaluated according to:

- the network topology (**TW**);
- the hydraulic resistance of the links (**HW**).

In particular, for each pair of nodes ( $i,j$ ) in the network, the corresponding entry in the first array is obtained as follows:

$$\mathbf{TW}(i, j) = \begin{cases} 0 & (i, j) \text{ is pump/valve or } i = j \\ \varphi \cdot \frac{Q_{ij}}{\max_{ij} \{Q_{ij}\}} + (1 - \varphi) \cdot \frac{\Delta z_{ij}}{\max_{ij} \{\Delta z_{ij}\}} & (i, j) \text{ is open pipe and } i \neq j \\ \infty & \text{otherwise} \end{cases} \quad (4.19)$$

where  $Q_{ij}$  is the demand served by the link connecting the nodes  $i$  and  $j$ , while  $\Delta z_{ij}$  is the absolute difference between the ground elevations of the same two nodes. Both the quantities are normalised by dividing by their maximum values and then combined with the blending factor  $\varphi$ .

Of course, if a pair of nodes is not connected by any link, the corresponding distance is set equal to infinity. The same is assumed if the nodes are connected through a closed pipe, which status must be preserved in the partitioned network. On the contrary, the value  $Q_{ij}$  can be obtained by considering the water

demands ( $Q_i, Q_j$ ) at nodes  $i$  and  $j$  and the numbers of links connected to them ( $m_i, m_j$ ):

$$Q_{ij} = \frac{Q_i}{m_i} + \frac{Q_j}{m_j} \quad (4.20)$$

Finally, a null weight is assumed for active (i.e. not closed) pumps and valves in order to consider the corresponding links as individual elements to be handled as a whole.

Similarly, the typology of the link is taken into account for evaluating the entries of the **HW** matrix. As for the open and existing pipes, the hydraulic resistance ( $res$ ) is considered, which depends on the pipe length ( $L$ ), diameter ( $D$ ) and roughness ( $R$ ). Since the values of this latter belong to its physical meaning (friction factor vs. hydraulic conductivity), the mathematical formulation of  $res$  should be addressed case-by-case according to the head-loss formula adopted for the hydraulic simulation. Without loss of generality, a definition based on the hydraulic conductivity is reported in Eq. 4.21.

$$\mathbf{HW}(i, j) = \begin{cases} 0 & \text{if } (i, j) \text{ is pump/valve or } i = j \\ res = \frac{L}{RD} & \text{if } (i, j) \text{ is open pipe or CV and } i \neq j \\ \infty & \text{otherwise} \end{cases} \quad (4.21)$$

The controlled direction of flow in the pipes is taken into account, as well. In brief, when a pipe is equipped with a check valve (CV) that constrains flow in the direction from  $i$  to  $j$ , the entry  $\mathbf{HW}(i, j)$  is considered equal to the hydraulic resistance of the pipe, while the symmetric one (with respect to the main diagonal of the matrix) is set to infinity.

Furthermore, it must be highlighted that the hydraulics of the pumps and valves is not properly considered in the definition provided in Eq. 4.21. However, even though a more detailed formulation could be introduced, this

methodological assumption is not expected to significantly affect the subsequent evaluations that are described in the next subsection.

### **4.3.2. Generation of network partitioning designs**

In this section, the procedure for obtaining the design of the network partitioning starting from a given set of values for the decision variables is discussed.

First, the number of DMAs in the current solution ( $NDMA$ ) is considered. As explained before, this value can range between 1 and the chosen upper bound ( $NDMA_{max}$ ). Of course, the solution corresponding to one DMA is equivalent to the non-sectorization of the network and all the subsequent evaluations would lead to no closure of network pipes.

Otherwise, the first  $NDMA$  genes representing the node indices in the current solution are adopted as the indices of the centroids of the DMAs to be designed. Then, starting from these nodes, the DMAs are created by adding all the other network nodes to the closest centroid one by one.

To this aim, and in order to meet the criteria discussed in Section 4.1, the “topological distance” between the nodes is considered. This distance is set equal to the length of the shortest-path from the centroid to the selected node, which is evaluated according to the weights in the matrix of Eq. 4.19.

The Floyd-Warshall Algorithm (FWA, see Section 4.3.3) is adopted for computing the shortest-paths between all pairs of nodes, whose lengths are stored in the following  $n \times n$  matrix ( $n$  is the number of network nodes, disregarding the node type):

$$\mathbf{TD}(i, j) = \begin{cases} 0 & \text{if } i = j \\ \text{shortest-path (FWA)} & \text{if } i \neq j \end{cases} \quad (4.22)$$

The steps to follow for the assignment of the nodes to the DMAs are reported in the pseudo-code in Fig. 4.3. For each node, a temporary distance ( $dist$ ) is

initialized to infinity. Then, for all of the DMAs in the current solution, this distance is compared to the length of the shortest-path from the node to the relevant centroid. If a lesser value is obtained, *dist* is updated and the corresponding DMA index is stored. Conversely, *dist* is preserved and the next DMA is considered.

<b>For</b> $i = 1$ <b>To</b> $n$	repeat for every network node
$dist = \infty$	initialize dummy distance
<b>For</b> $d = 1$ <b>To</b> $NDMA$	repeat for every DMA
$j = \mathbf{centroid}(d)$	select the DMA centroid
<b>If</b> $TD(i, j) < dist$	if a lesser distance is found
$dist = TD(i, j)$	update the distance
$\mathbf{dma}(i) = d$	store the DMA index
<b>End If</b>	
<b>Next</b> $d$	
<b>Next</b> $i$	

Fig. 4.3 – Assignment of the network nodes to the DMAs

It is useful to remark that if the graph of the network is connected, the path between every pair of nodes surely exists and its length is a finite value. Therefore, at the end of the process, every node is assigned to a DMA. For the same reason, the adopted procedure ensures the internal connectivity of the DMAs, so that no subsequent verification is required.

Furthermore, it is possible to provide a more detailed explanation about the meaning of the blending factor. Since the number of DMAs and the centroid indices are integer and bounded variables, similar combinations of them are likely to be reproduced at different generations of the optimization algorithm. The use of the blending factor allows the shapes and the sizes of the DMAs to change, even though equal numbers of DMAs and the same sets of centroids are adopted. Although a more proper characterization of this interesting aspect

should be addressed with a specific numerical investigation, it is expected to result in an increased differentiation of the solutions and a more efficient exploration of the objective space.

Moreover, according to the provided definition of “topological distance” and to the expressions in Eq. 4.19, the distance between two nodes increases with the sum of their water demands and with the difference between their ground elevations. Consequently, through the minimization of the shortest-paths, for low values of  $\varphi$  the algorithm should lead towards solutions characterized by the grouping of nodes with similar elevations. Conversely, assuming a homogeneous distribution of the water demands in the network, when higher  $\varphi$  are adopted, the end nodes of very long pipes are separated into different DMAs (by closing the link connecting them). This results in a limitation to the sizes of the DMAs and, therefore, in a decreased probability of violating the corresponding constraint.

Once a new solution is created, it is brought into the hydraulic simulation model by closing the pipes which connect different DMAs. However, after this operation some DMAs might become completely isolated from the water sources, as depicted in Fig. 4.4a. In this case, the supply must be provided by opening the most appropriate connection link between different DMAs (cascade supply, Fig. 4.4b).

The solution to this problem requires the exploration of a domain whose dimension rapidly increases in relation to the number of boundary nodes of the two facing DMAs (combinatorial explosion). Moreover, this operation has a great influence on the hydraulic response of the partitioned network. Very low pressures could easily affect the isolated DMA if there is not sufficient hydraulic head preserved at its entry point. With the aim of tackling both the mentioned issues, the procedure represented in Fig. 4.5 is applied.



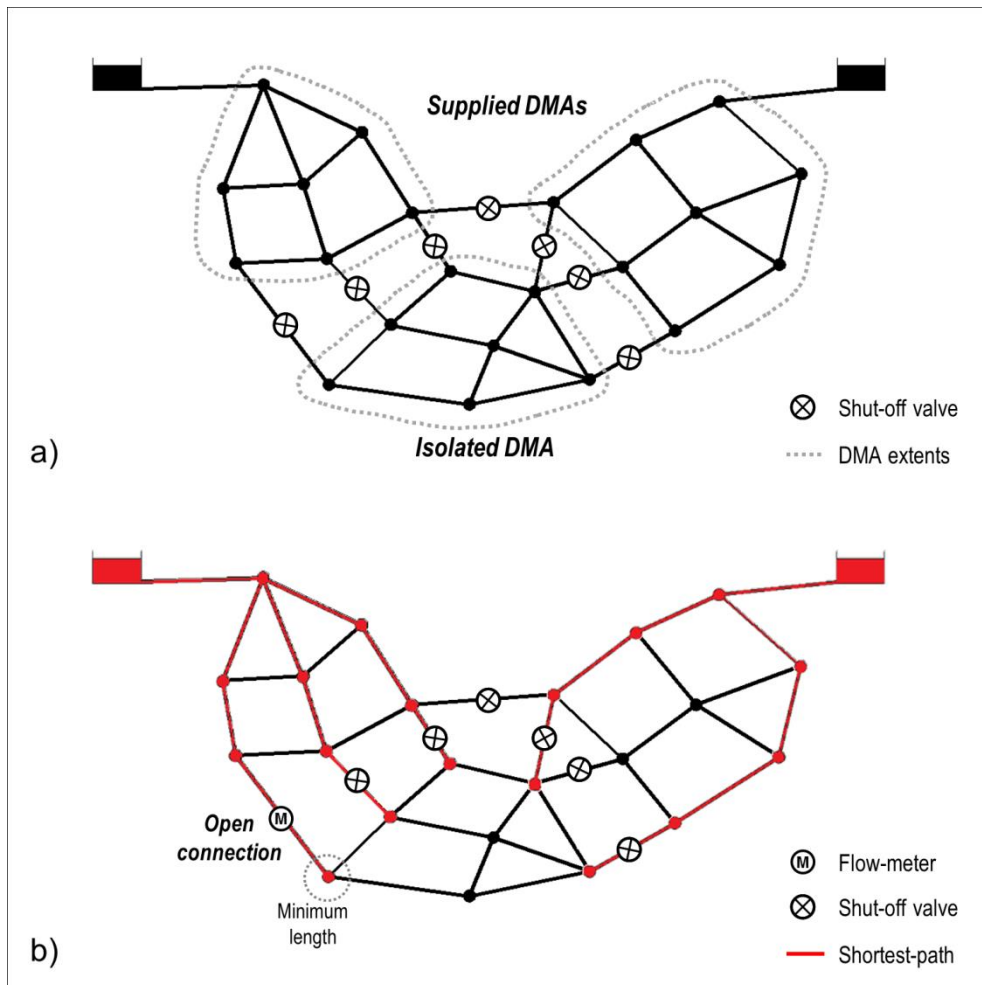


Fig. 4.4 – Setting of the boundaries of the DMAs and cascade supply

First, the presence of water sources among the nodes of each DMA is checked. If this condition is verified, the DMA is directly supplied and it is included in the set  $SupD$ ; otherwise, it is an isolated DMA and it is assigned to the set  $unSupD$ .

Then, for the first element ( $\delta$ ) in the latter set, the adjacent DMAs are considered, namely the DMAs having at least one node connected to a node in  $\delta$  (through a closed pipe). If there are not any adjacent supplied DMAs, the DMA is skipped and queued, and the next  $\delta$  is analyzed. Conversely, the adjacent DMAs are included in the set  $adjD$ , while the boundary nodes of  $\delta$  connected to the DMAs in  $adjD$  are assigned to the set  $boundD$ .

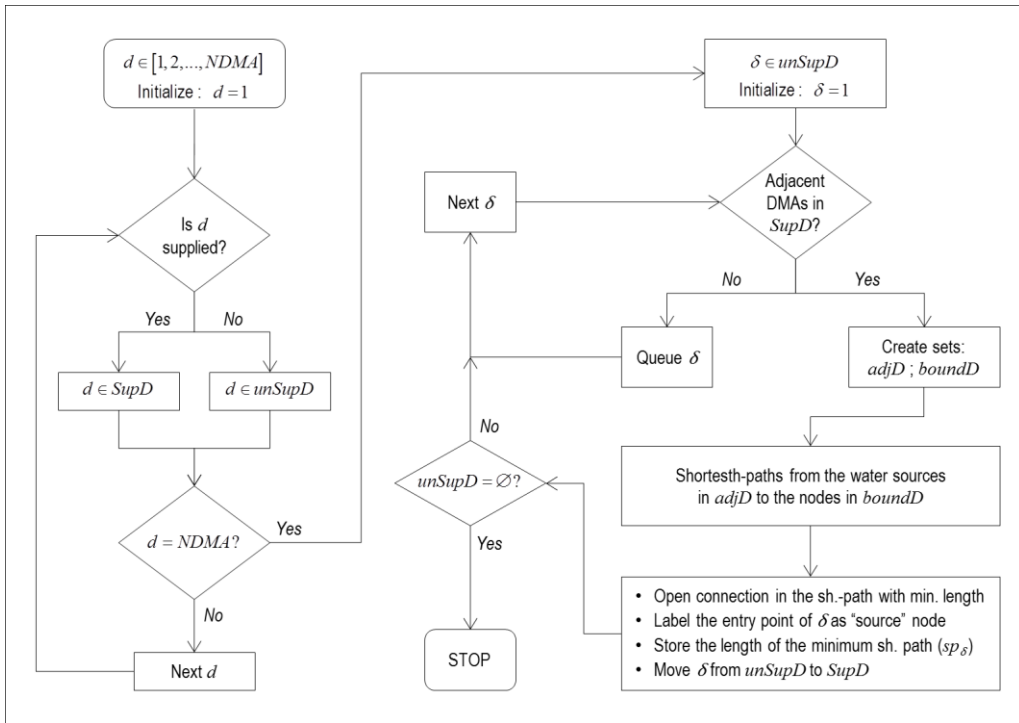


Fig. 4.5 – Procedure for opening the best connections between DMAs

Then, the FWA is used again for evaluating the shortest-paths from the water sources in the DMAs of  $adjD$  to the nodes in  $boundD$ . However, in order to ensure the maximum possible hydraulic head at the entry point of the isolated DMA, the weights on the graph links are set equal to the quantities in Eq. 4.21.

Next, the following operations are performed: i) the connection belonging to the shortest-path with minimum length (Fig. 4.4b) is opened; ii) the entry node of  $\delta$  (the boundary node of  $\delta$  belonging to the shortest-path) is labeled as “source” node; iii) the length of the shortest-path ( $sp_{\delta}$ ) is stored; iv)  $\delta$  is moved from  $unSupD$  to  $SupD$ .

The described sequence is repeated until  $unSupD$  is emptied. Furthermore, when calculating the shortest-paths from the source nodes (the entry nodes) of the indirectly supplied DMAs, the length of the shortest paths must be increased of the quantities  $sp_{\delta}$ .

### 4.3.3. Floyd-Warshall algorithm

The Floyd-Warshall Algorithm (FWA) is a computational method for finding the shortest-paths between all pairs of nodes in an oriented and weighted graph  $G(V,E)$ . Robert Floyd published it in its most commonly recognized form in 1962, and it is an example of dynamic programming.

Several approaches can be found in literature for the evaluation of the shortest-paths in a looped network. Among these, the FWA shines for its reasonable complexity (the worst case performance is  $O(n^3)$ ,  $n$  being the number of vertices) compared to its very simple formulation. Moreover, unlike other well-known methods, such as the Dijkstra algorithm (Dijkstra 1959), the FWA can be applied to graphs with either positive and negative edge weights (but with no negative cycles, i.e. cycles whose edge weights sum to negative values).

The FWA (Fig. 4.6) operates by incrementally improving an estimate of the shortest-path between two vertices, until the estimate is optimal. In its simplest form, it requires the storage of a single  $n \times n$  array (**SP**) containing the lengths of the shortest-paths between each pair of vertices. However, in order to preserve the information about the paths, another  $n \times n$  array (**P**) can be considered, whose entries  $\mathbf{P}(i,j)$  correspond to the last predecessors in the shortest-path from vertex  $i$  to vertex  $j$ .

The **SP** array is initialized by assigning to each element  $\mathbf{SP}(i,j)$  the weight of the graph edge connecting  $i$  and  $j$ , while in **P** all the columns are identically set equal to the list of the row indices from 1 to the number of vertices in the network. Of course, all the entries on the main diagonal of **SP** are null, while those corresponding to non-existing edges are set equal to infinity.

The basic idea of the FWA is to make a recursive comparison between the direct path from a source node  $i$  to an end node  $j$  (i.e. the weight on the graph edge between the nodes) and the concatenation of the paths from  $i$  to an intermediate node  $k$  and from  $k$  to  $j$ , where  $k$  is varied among all the other vertices in the graph.

```
# Initialize arrays
For i = 1 To n
  For j = 1 To n
    SP(i,j) = weight(i,j)           initialize shortest-path length
    P(i,j) = predecessor(i,j)      initialize predecessor
  Next j
Next i

# Run the FWA
For k = 1 To n
  For i = 1 To n
    For j = 1 To n
      If SP(i,j) > SP(i,k) + SP(k,j)  compare lengths
        SP(i,j) = SP(i,k) + SP(k,j)  update shortest-path length
        P(i,j) = k                    update predecessor
      End If
    Next j
  Next i
Next k
```

*Fig. 4.6 – Floyd-Warshall Algorithm*

If the concatenated path is more convenient, the corresponding **SP** element is updated and the predecessor  $k$  is stored in **P**; otherwise, both the original values are preserved. This recursive analysis is realized through the three nested for-loops in Fig. 4.6, which represent the heart of the FWA.

#### 4.3.4. NSGA-II algorithm

In the proposed approach, the NSGA-II algorithm is adopted for solving the multi-objective optimization problem.

NSGA (Non-Dominated Sorting Genetic Algorithm) is a very popular Multi-Objective Evolutionary Algorithm (MOEA) firstly introduced by Srinivas and Deb in 1994. However, since this version has been generally criticized for its computational complexity, the lack of elitism and the need of specifying a sharing parameter for ensuring a sufficient diversity in the population, an improved version was proposed, namely the NSGA-II (Deb et al. 2002).

This algorithm (also named “fast non-dominating sorting approach”) uses a very efficient mechanism for sorting a population into different non-domination levels. A very simple way to perform this ranking is to compare all of the solutions to find the non-dominated ones in each level, which involves in the worst case  $O(MN^3)$  computations (where  $M$  is the number of objectives and  $N$  is the size of the population). Instead, in NSGA-II the following entities are evaluated for each solution  $p$ : the number of solutions that dominate  $p$  (domination count,  $n_p$ ) and the set ( $S_p$ ) of the solutions dominated by  $p$ .

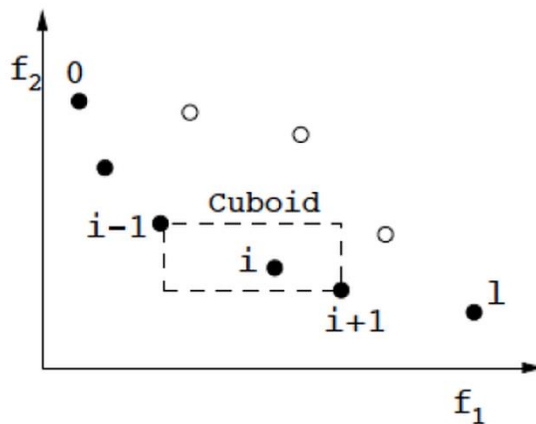
Of course, all the solutions with a null domination count constitute the first non-dominated front. For each of them, all the members  $q$  of the dominated set  $S_p$  are visited, and their domination counts are reduced by one. Then, having reached a domination count equal to zero, the members are recognized as belonging to the second non-dominated front. The procedure is continued until all fronts are identified, and this is performed with  $O(MN^2)$  complexity in the worst case.

Another important feature of NSGA-II is its capacity to preserve an adequate diversity (i.e. a good spread) in the obtained set of solutions. In the context of multi-objective optimization, this is as important as the convergence to the Pareto-optimal set. To this aim, in NSGA-II the crowded-comparison is adopted. This approach consists of comparing the solutions in the same non-

domination level on the basis of a distance measure, which depends on the proximity with other solutions.

The computation of the so-called *crowding-distance* requires the preliminary sorting of the population according to the ascending magnitude of each objective function. Next, an infinite distance value is assigned to the boundary solutions (those characterized by the minimum and the maximum values, respectively). All the other intermediate solutions are assigned a distance value equal to the absolute normalized difference in the objective function values of two adjacent solutions. This calculation is continued with other objectives, and then the overall crowding-distance value is obtained as the sum of the individual distances evaluated for each objective.

In other words, the crowding-distance of a solution is as an estimate of the perimeter of the cuboid having as vertices the nearest neighbours in its front (see Fig. 4.7). Therefore, solutions with larger crowded-distance are preferred, as it increases with the spread of the solutions within the non-domination level.



*Fig. 4.7 – Crowding-distance calculation (Deb et al. 2002)*

The described procedure is used for non-constrained optimization problems. However, constrained problems like the one proposed in this thesis are very common in practice, but not much attention has been paid so far in this respect among the EA researchers (Deb et al. 2002).

Most of the proposed approaches use penalty functions for handling the constraint violations. These penalties are often added to the objectives, so that when a feasible solution is compared with an infeasible one, the latter is discarded because of the worse value of the objective function. Nevertheless, this may lead to inefficient evaluations when the magnitude of the objectives is not known a-priori.

In NSGA-II the constraint-handling is performed more efficiently through a slight modification in the definition of dominance. In particular, a solution  $i$  is said to constrained-dominate a solution  $j$ , if any of the following conditions is true:

- solution  $i$  is feasible and solution  $j$  is not;
- solutions  $i$  and  $j$  are both infeasible, but solution  $i$  has a smaller overall constraint violation;
- solutions  $i$  and  $j$  are feasible and solution  $i$  dominates solution  $j$ .

Consequently, the feasible solutions are always assigned a better non-domination rank compared to infeasible ones, while within the set of feasible solutions the ranks are assigned according to the objective function values. Moreover, this approach makes it possible to rank the infeasible solutions, which are compared on the basis of the constraint violations. As a result, the efficiency of the algorithm is increased, even though its computational complexity is not affected.

The different steps of the algorithm are now described. First, the NSGA-II is initialized with the random generation of an *initial population*, that is an array having a number of rows equal to the size of the population (*popsize*) and a number of columns equal to the number of decision variables (*nvars*). The former must be specified among the NSGA-II options, while in the proposed methodology the latter depends on the maximum number of DMAs adopted for the analysis ( $NDMA_{max}$ ). In addition to the indices of the centroids (in number

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equal to  $NDMA_{max}$ ), the number of DMAs actually considered in the solution and the blending factor are required. Therefore,  $nvars$  corresponds to the quantity  $NDMA_{max}+2$ .

Then, every *chromosome* (i.e. each row in the population array) is subjected to the generation phase (described in Sec. 4.3.2) and to the evaluation of the objectives and of the constraint violations. This makes it possible to carry out a preliminary non-dominated sorting and the evaluation of the crowding distances, which are used for ranking the solutions. Next, the main loop of the algorithm is executed, which consists of the following steps (see Fig. 4.2):

- **Selection:** in this phase, the parent solutions are selected for generating a child population. Among the different possibilities, the Tournament Selection (TS) is adopted here because it is easier to code, has good computational performance and provides the option to adjust the selection pressure (Miller and Goldberg, 1995). The TS operates by extracting from the original population a random subset of  $k$  solutions, which are compared on the basis of the constrained-non-domination and the crowding distance. The winner of each tournament is considered as a candidate parent; of course, the best-ranked chromosomes are likely to have more copies of their *genes* (i.e. their subsets of entries) in the child population. This operation is repeated until the mating pool is filled (which has half the population size). The most popular choice for the tournament size is  $k = 2$  (Binary Tournament Selection), and the same is assumed in this case.
- **Reproduction:** the genetic operators are used for creating an offspring population from the parent solutions. In the proposed methodology, the Simulated Binary Crossover (SBX, Deb and Agrawal 1995, Deb and Kumar 1995) and the Polynomial Mutation (PM, Raghuvanshi and Kakde 2004) are adopted. Rounding functions are introduced for handling the integer variables. The parameters of the genetic operators (*Distribution Index for Crossover*,  $\eta_x$ , and *Distribution Index for Mutation*,  $\eta_m$ ) must be specified in



the initialization phase of the optimization algorithm. Large values of these parameters involve less spread of the children solutions towards the parents. Therefore, due to the presence of integer variables, low values of  $\eta_x$  and  $\eta_m$  should be assumed, since this would improve the efficiency in the exploration of the solutions domain (i.e. the capacity of avoiding local minima is increased). The reproduction is performed by randomly selecting the parents within the mating pool; then, the SBX and the PM are applied with respective probabilities  $Xover$  and  $(1 - Xover)$ , with  $Xover$  being the *crossover rate* (set equal to 0.9). At the end of this step, an *intermediate population* is created by combining the parent solutions with the offspring;

- **Evaluation:** the children solutions are passed to the algorithm of Fig. 4.5 in order to create the corresponding WDN partitioning designs and, therefore, to evaluate the objective functions and the constraint violations;
- **Non-dominated sorting and crowding-distances evaluation for the intermediate population:** the intermediate population is sorted according to non-domination, and the crowding-distances are evaluated. It is worthwhile noting that, since all the previous and the new population members are included in the intermediate population, the *elitism* in the optimization process is ensured;
- **Refinement:** this process is required for restoring the original size of the population. To this aim, the first level of non-domination ( $F_1$ ) is considered. If the size of  $F_1$  is smaller than *popsiz*e, all its members are included in the new population. The same is done for the subsequent fronts ( $F_2, F_3, \dots$ ) until no more sets can be entirely accommodated. Then, two cases are possible: i) the sum of the sizes of the selected fronts is equal to *popsiz*e; ii) the new population can still accommodate solutions. If the first is verified, no more evaluation is required and the process stops. Otherwise, the procedure depicted in Fig. 4.8 is applied: the first excluded front ( $F_L$ ) is considered, and

its members are sorted by descending crowding-distance. The best ones are then selected for filling the remaining slots in the new population.

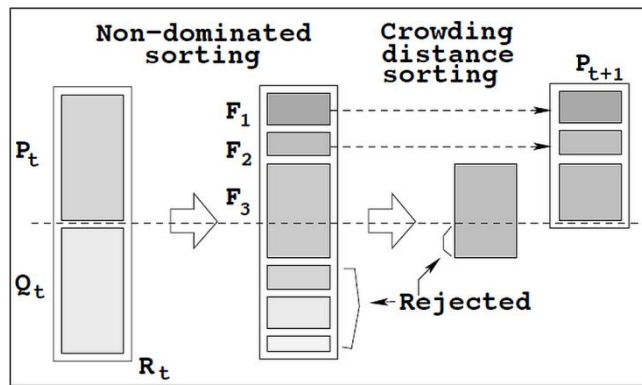


Fig. 4.8 – Refinement procedure in NSGA-II (Deb et al. 2002)

The described steps are repeated until the desired number of generations ( $ngen$ ) is reached. Of course, this parameter must also be provided among the NSGA-II options.

At the end of the process, the solutions in the first non-dominated level represent an estimate of the true and unknown Pareto front, and then they can be considered as optimal designs of the DMAs. In order to improve the convergence towards the global optimum, large values for  $popsiz$  and  $ngen$  should be assumed. However, the negative effect on the computational time must be carefully taken into account, since the number of evaluations increases with the product of these parameters.

Furthermore, according to the theory of evolutionary algorithms, in order to improve the performance several runs should be performed starting from different random seeds (i.e. the randomly generated initial populations).

### **4.3.5. Hydraulic simulator**

The evaluation of the objective functions and of the constraint violations requires information about the water pressures before and after the introduction of the DMAs. Their values must satisfy the hydraulic equations mentioned in Sec. 4.2.2. Therefore, hydraulic simulations must be performed on the network model.

To this aim, most of the recent methodologies for the optimization of WDNs adopt EPANET (Rossman, 2000), the hydraulic simulator provided by the American Environmental Protection Agency (EPA). EPANET is a free software program which performs extended period simulations about the hydraulics and the water quality in pressurized networks. It is successfully used for many applications, such as sampling design, hydraulic model calibration and evaluation of risk scenarios (i.e. fire service, pipe failures, contamination). EPANET can also compute the pumping energy and cost while being able to handle different types of valves, including shut-offs, check, pressure regulating and flow control.

EPANET solves the hydraulic equations through the use of the “gradient method” proposed by Todini and Pilati (1987). It computes head-losses by using one of the following formulas: i) Hazen-Williams (the most commonly adopted in the U.S.), which is used to model full flow of water under simplified conditions (turbulent flow, temperature around 15 degrees Celsius); ii) Darcy-Weisbach, the most theoretically correct one, that can be applied to all fluids and flow regimes; iii) Chezy-Manning, which uses the Chezy’s roughness coefficients for the Manning’s equation (most used for open channel flows). EPANET can also handle minor losses, pump curves (for both head and efficiency) and time patterns, which can be applied to every network component (e.g. water level in tanks, customer demands, pump and valve scheduling).

The EPANET computational engine can be easily incorporated as a module in larger analysis programs through its dynamic link library (DLL) of functions,

namely the EPANET Programmer's Toolkit. Different updates or extensions of this library have been introduced for overcoming the main limitations of this hydraulic simulator.

Among these, the EPANETpdd extension (Morley and Tricarico 2008) is selected in this work. The major novelty introduced by EPANETpdd is the implementation of the Pressure Driven Demand (PDD) analysis, which is not supported in the original version of the EPANET Toolkit. Therefore, pressure-dependent demands can be modelled, as well as pressure-dependent leakages. The required parameters can be specified node-by-node, while the emitter coefficients in intermediate conditions (i.e. when there is not enough pressure to completely satisfy the node demand) are dynamically calculated according to the demand specified in the input file. As a result, it is easier to perform the conversion of a model between demand-driven and pressure-driven mode.

However, since the proposed methodology aims at the preservation of the minimum service pressure (i.e. the critical pressure in EPANETpdd), the PDD analysis for water demands is not strictly required. Conversely, great attention is paid to the water leakages, because they affect both the objective functions. In EPANET the pressure-dependent leakages are modelled through emitter nodes, which deliver a variable demand depending on the water pressures. When negative pressure occurs (i.e. when the total head is below the node elevation) the corresponding water leakage should be equal to 0. Instead, EPANET calculates a negative demand (i.e. supply), thus biasing the results of the analysis. This feature was fixed in EPANETpdd, and therefore it is adopted here as hydraulic simulator. Furthermore, EPANETpdd allows the user to get additional returns from the network model (e.g. the coordinates of the nodes and of the vertices).

#### **4.4. Technical evaluation of the solutions**

At the end of the optimization process an approximation of the true Pareto front is obtained. Each solution in this front represents a feasible and optimal solution for the design of the WDN sectorization. The decision-maker is then allowed to select the best one according to his preferences; obviously, the simplest choices consist of the solutions corresponding to the minimum values of the objective functions (minimum Total Cost or minimum Resilience Deviation Index).

However, the Pareto front can include a very large number of intermediate solutions, each one representing an optimal trade-off between the objectives. In order to provide further support to the decision through a more detailed understanding of the obtained results, the evaluation of a number of technical indicators is proposed.

First of all, the actual number of DMAs per each solution is taken into account. Given the unavoidable uncertainties related to the modelling of WDNs, the effects of the sectorization on the system operation could diverge from those obtained through the analysis. Therefore, particular attention must be paid in the practical implementation of the solutions. To this aim, the ones characterized by less DMAs could be preferred, since this would facilitate minor adjustments for resolving unexpected effects.

For the same reason, the number and the diameters of the intercepted pipes are also considered. In addition, when the shut-off valves for closing the pipes are not already installed, construction works must be carried out in the water network. The magnitude of these interventions increases with the number and the diameters of the interested pipes. Therefore, especially in strongly urbanized contexts, the closure of a larger number of small pipes instead of few large pipes can be preferred.

The financial evaluation of the solutions is already performed through the Total Cost Function (Eq. 4.1). However, one can be interested in the discrimination between the short term investment (i.e. the construction cost) and

the cost saving achievable in the long term because of the reduction of the water leakages. Moreover, while the former can be assessed with sufficient accuracy, the evaluation of the latter is affected by the uncertainties due to the modelling of the pressure-dependent leakages and to the actual hydraulic response of the partitioned network.

#### **4.5. Example application**

With the aim of providing further insight about the proposed approach, the application of the methodology to a well-known case study from literature is presented here, namely the 25-nodes network presented in Jowitt and Xu (1990), which was firstly introduced by Bargiela in 1994. This WDN (Fig. 4.9) consists of 37 pipes, 22 nodes and 3 reservoirs. The ground elevations of the nodes range from 7 to 23 meters above the ordnance datum, while the normal level in all the three reservoirs is approximately 54.6 m.

The lengths of the network pipes vary in a very large interval (from 300 to 5,150 meters), and 6 different diameters are adopted (152, 229, 305, 381, 457, 475 mm). A roughness coefficient is assigned to each pipe for the evaluation of the head-losses according to the Hazen-Williams formula. It must be specified that the roughness of pipes 8 and 10 are those reported in Creaco and Pezzinga (2014), where more appropriate values are introduced instead of the very low coefficients used by Jowitt and Xu (1990). Three flow-control valves are also introduced in the original example, but their presence is neglected in the present application.

The water demands are directly assigned to the nodes, and they vary during the day according to a given time pattern. The same happens for the total head of the reservoirs, whose depletions are related to the operative schedules of the respective filling pumps. The minimum service pressure for fulfilling the demand at nodes is fixed at 30 meters.

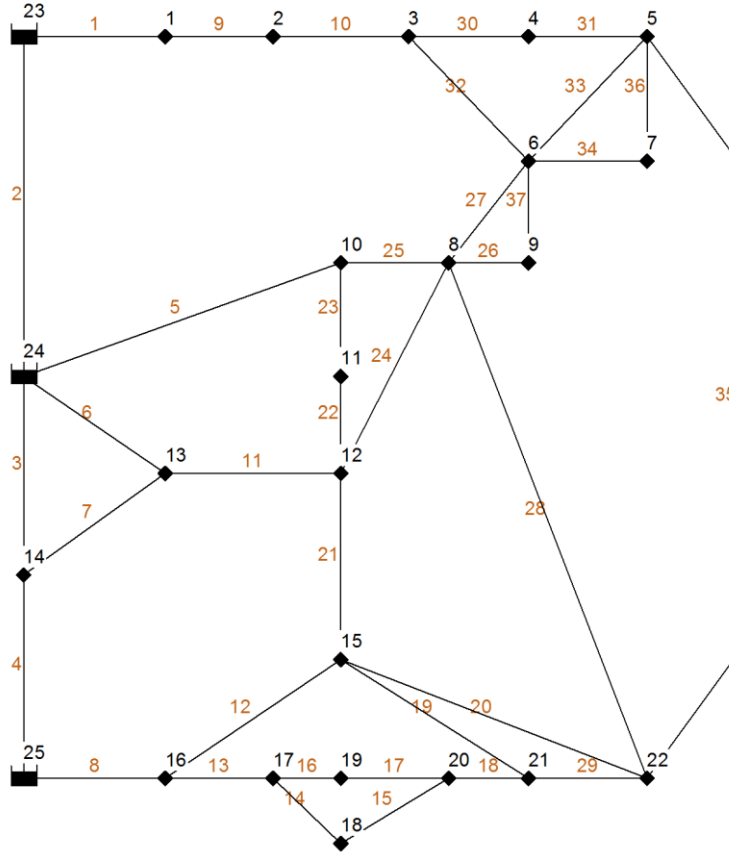


Fig. 4.9 – 25-nodes network (Jowitt and Xu, 1990)

Pressure-dependent leakages are introduced as well. However, in Jowitt and Xu (1990) they are assigned to the pipes, and not to the nodes. In particular, the following equation is considered for the evaluation of the water leakage volume ( $QL_{ij}$ ) for each pipe:

$$QL_{ij} = \beta_L L_{ij} \dot{P}_{ij}^{1.18} \quad (4.23)$$

where  $L_{ij}$  is the length of the pipe connecting the nodes  $i$  and  $j$ ,  $\dot{P}_{ij}$  is the average pressure along the pipe (i.e. the mean of the pressure values at  $i$  and  $j$ ), and  $\beta_L$  is a constant coefficient assumed equal to  $10^{-5}$  for all the pipes. However, since the adopted hydraulic simulator (see Sec. 4.3.5) requires the assignment of the water leakages at nodes, the equivalent emitter coefficients ( $\alpha_i$ ) should be derived from Eq. 4.23:

$$\begin{aligned}
 Q_i &= \alpha_i P_i^{1.18} = \sum_{j=1}^N \frac{Q L_{ij}}{2} = \sum_{j=1}^N \frac{\beta_L L_{ij} \dot{P}_{ij}^{1.18}}{2} \\
 \dot{P}_{ij} &\approx P_i \Rightarrow \alpha_i P_i^{1.18} \approx \frac{\beta_L P_i^{1.18}}{2} \sum_{j=1}^N L_{ij} \Rightarrow \alpha_i \approx \frac{\beta_L}{2} \sum_{j=1}^N L_{ij}
 \end{aligned} \tag{4.24}$$

where  $Q_i$  is the water leakage at the  $i$ -th node,  $P_i$  is the water pressure and  $N$  is the number of network pipes linked to  $i$ . This assumption involves an increase in the total supplied volume under uncontrolled conditions of approximately 20% (with respect to water demand) that is consistent with the value assumed in other works.

The application of the proposed methodology to this case study also requires an estimate of the total number of customer connections in the network (*TotCon*). To this aim, the procedure mentioned in Sec. 4.3.1 is adopted. The total daily supplied volume (i.e. the sum of the water demand at nodes over time) is 13,230 m<sup>3</sup>. Assuming a per-capita consumption of 200 l/ab/d (litres per inhabitant per day) and an average value of three inhabitants per customer, *TotCon* is estimated in approximately 1,900 connections.

Among the analysis options, given the small size of the network, the upper limit for the number of DMAs is set to 3. A budget limit of 10,000 € and an annual discounting rate equal to 0.03 are considered. Equal emphasis is given to the required investment and the cost of the water leakages in the Total Cost Function ( $w_B = w_L = 0.5$ ).

The unit costs of the shut-off valves and of the flow-meters for every pipe size are reported in Tab. 4.2, while for the water leakages a cost of 0.15 €/ m<sup>3</sup> is assumed, which is consistent with the wholesale price of purchasing of water. It must be highlighted that the adopted values are only intended to provide a realistic combination of parameters for the evaluation of the objectives, and that there is no claim of obtaining exportable results without a more in-depth characterization of the same features.



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Pipe diameter (mm)	Shut-off valve (€)	Flow meter (€)
152	245.00	2,020.00
229	865.00	2,580.00
305	1,930.00	3,345.00
381	3,450.00	4,315.00
457	5,420.00	5,490.00
475	5,955.00	5,800.00

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Tab. 4.2 – Unit costs for the 25-nodes network

#### 4.5.1. Calibration of NSGA-II

The reduced size of the network and its very simple structure make it possible to perform several runs of the optimization algorithm in reasonable time. Therefore, this application is used for the calibration of the NSGA-II parameters that were not previously specified, namely the population size (*popsiz*e), the number of generations (*ngen*), the distribution index for crossover ( $\eta_x$ ) and the distribution index for mutation ( $\eta_m$ ).

To this aim, a large number of test cases are analyzed, each one consisting of a different combination of the above mentioned parameters. For every parameter, three different levels are taken into account, as reported in Tab. 4.3. Therefore, 81 test cases are considered, and for each of them five runs are performed. This results in a total of 405 different tests.

However, in order to ensure the best convergence towards the true Pareto front, further tests are introduced in which very large values for *popsiz*e and *ngen* are adopted (respectively 100 and 1,000), while the parameters  $\eta_x$  and  $\eta_m$  are varied as before. Given the very large number of evaluations to be performed, in this case the number of runs was reduced to 3. Consequently, 8 more test cases (27 tests) are considered. Then, the total number of considered test cases amounts to 90, while the total number of different tests is 432. The detailed test plan is reported in Appendix A, and it involves 3,429,000 evaluations of the objective functions and of the constraint violations.

<b>Test cases</b>	1 to 81	82 to 90
<b>No. of runs</b>	5	3
<b>Parameter</b>	<b>Values</b>	
<i>Population size</i>	10 ; 30 ; 50	100
<i>Number of generations</i>	30 ; 50 ; 100	1,000
<i>Distr. Index for Crossover</i>	1; 10 ; 20	1; 10 ; 20
<i>Distr. Index for Mutation</i>	1; 10 ; 20	1; 10 ; 20

Tab. 4.3 – Calibration of NSGA-II. Summary of the tested parameters

### 4.5.2. Algorithm performance evaluation

The evaluation of the performance of the designed algorithms is carried out according to the methodology proposed in Wang et al. (2014). This procedure is based on the evaluation of the Normalized Hyper-Volume (*NHV*), which is a single metric that can efficiently assess both the aspects of the multi-objective algorithm performance, namely the convergence and the diversity (Deb 2011).

The *NHV* is calculated as the ratio between the hyper-volume (*HV*) defined by a generic approximation set and the true Pareto front. The evaluation of the *HV* (Fig. 4.10) is performed by summing the volumes of the hyper-cubes constructed with a reference point (i.e. a vector with the worst objective values) and the solutions in the Pareto front as diagonal corners.

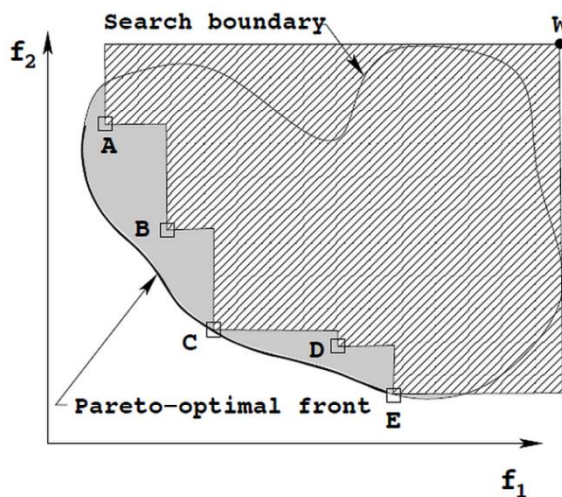


Fig. 4.10 – Evaluation of the hyper-volume (Deb 2011)

The generic approximation sets are obtained through the following procedure: for each of the presented test cases, a non-dominated sorting is performed on the best solutions obtained in the relative single runs (i.e. the solutions in the Pareto fronts of each run). This operation is referred here as “aggregation”. The *Aggregate-Pareto front* is the resulting set of solutions in the first level of non-domination, and it is considered as an approximation of the true (and unknown) Pareto front.

In order to perform the best estimate of the latter, that is required for the calculation of the *NHV*, a non-dominated sorting procedure is applied to the solutions of all the obtained *Aggregate-Pareto fronts*. Again, the solutions in the first level of non-domination are considered, as they represent the quasi-true Pareto front (*quasi-Pareto*). It is also worthwhile noting that the obtained set represents the best solution to the optimization problem.

### **4.5.3. Discussion of the results**

The tests described so far were automatically performed in the Matlab® (Mathworks) environment by using a PC with an Intel Core i5-4200U 2.30 GHz processor and 8 GHz RAM. The calculations were completed in approximately 27 hours, but the most of the test cases (from 1 to 81) were analyzed in just 5 hours and 25 minutes. The huge time requested by the analysis of the subsequent test cases was due to very large number of planned evaluations (2,700,000 of total 3,429,000).

The calculation of the *NHVs* makes it possible to rank the designed algorithms (i.e. the different test cases) in order to select the best combination of parameters for the NSGA-II. The best performances are obtained when large numbers of evaluations are performed; therefore, as expected, the increasing of the population size and of the number of generations contribute to the improvement of the algorithm performance.

However, the highest value of the *NHV* (0.9641) is provided by the combination of parameters adopted in the test case 75, which consists of the following:

- Population size: 50;
- Number of generation: 100;
- Distribution Index for Crossover: 1;
- Distribution Index for Mutation: 20;

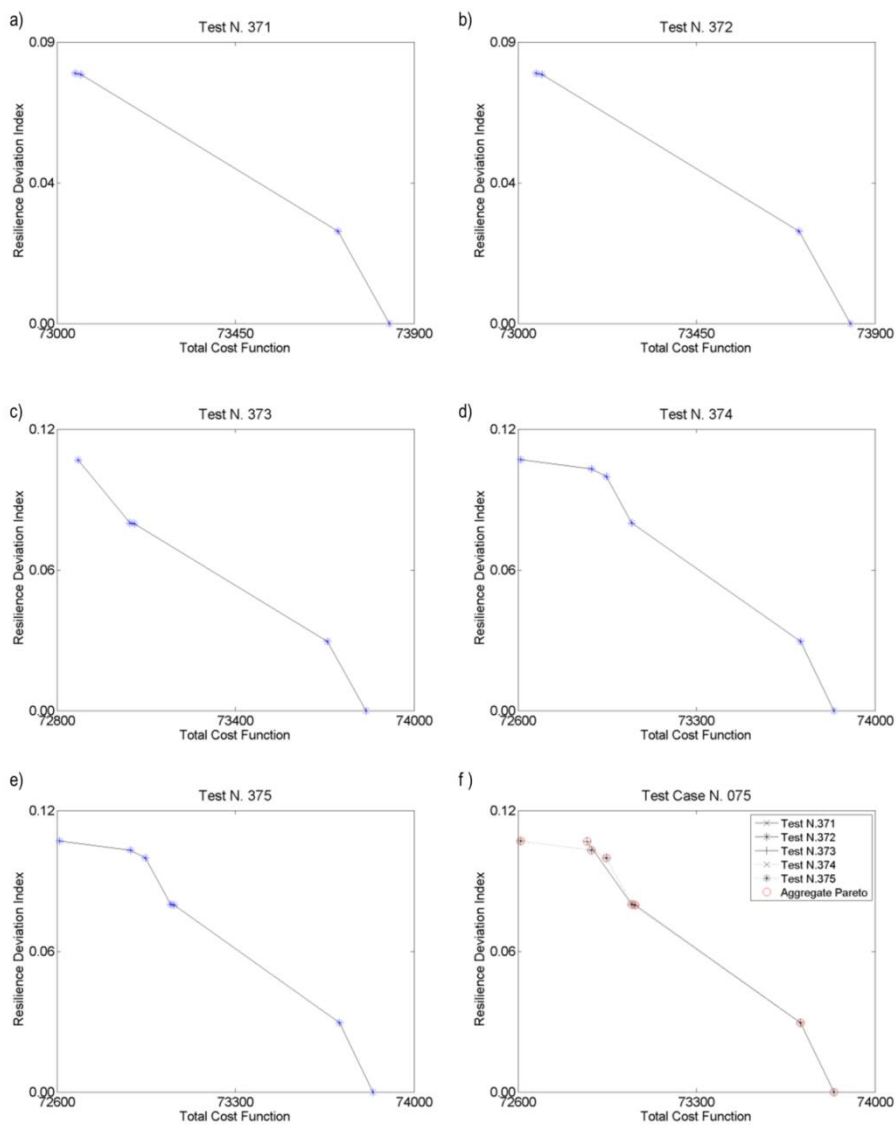


Fig. 4.11 – Test case No. 75: (a-e) Pareto fronts obtained in five runs; (f) aggregation of the solutions

The obtained  $NHV$  is significantly higher than the one of the second ranked algorithm (test case 85,  $NHV=0.7799$ ), in which the largest values of  $popsiz$ e and  $ngen$  are adopted (100 and 1,000), while  $\eta_x$  and  $\eta_m$  are 10 and 1, respectively.

The Pareto fronts obtained in the five runs of the test case 75 are reported in Fig. 4.11 (letters a-e). The extrapolation of the Aggregate-Pareto front is represented in Fig. 4.11f, which shows that all the obtained solutions have to be included in the optimal set, thus providing evidence of the need of running several times the optimization algorithm.

The quasi-Pareto front can be obtained by plotting together all the Aggregate-Pareto solutions in the objectives domain. The red line in Fig. 4.12 highlights the 9 points that are selected as the optimal solutions for the sectorization of the example network.

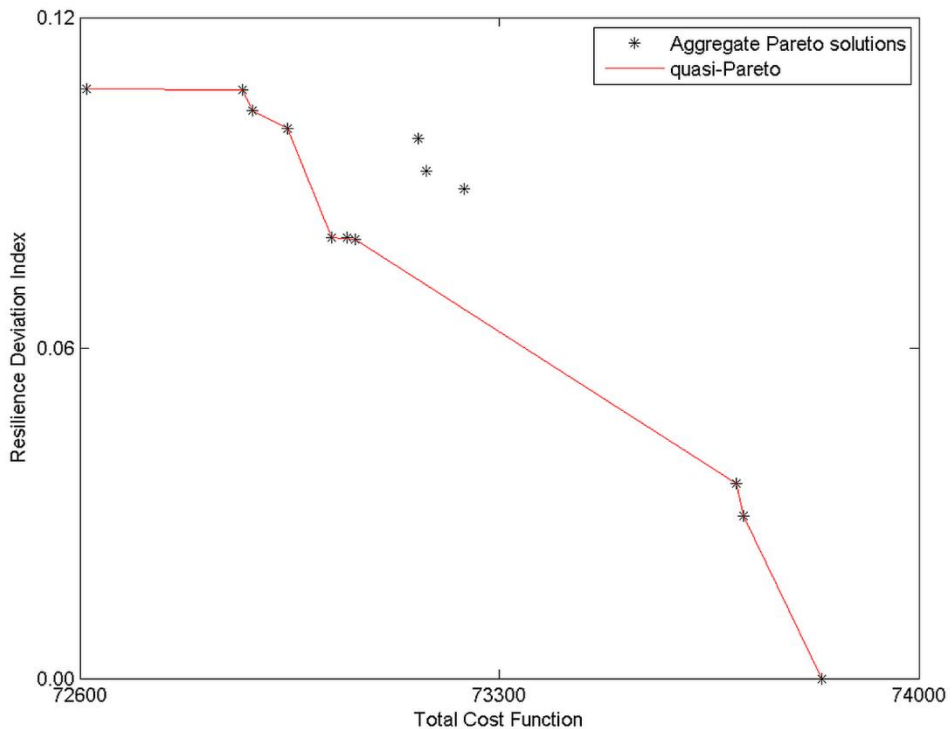
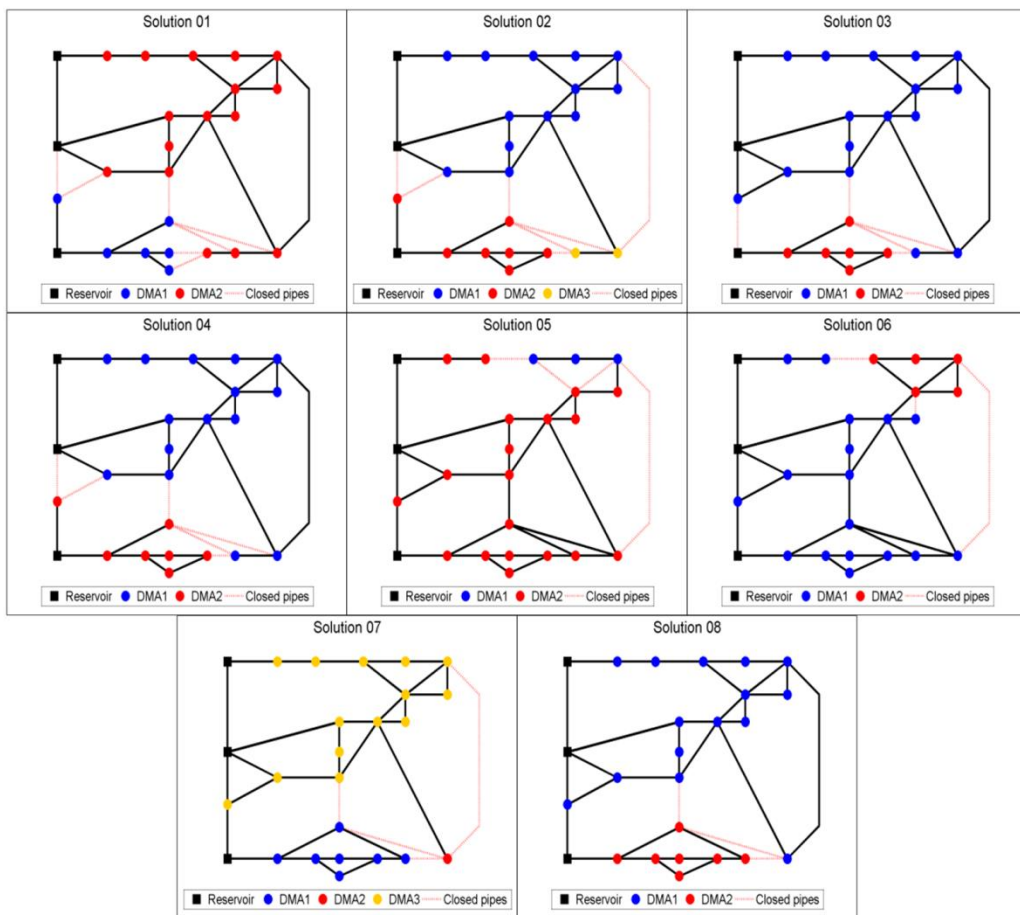


Fig. 4.12 – quasi-Pareto front for the 25-nodes network

The obtained values of the Total Cost Function range between 72,611 and 73,838. It is useful to point out again that, by virtue of the choice of the weights in the TCF, these values correspond to half the annual operational cost for the partitioned network.

As for the Resilience Deviation Index, a maximum value of 0.1070 is detected, which means that the reduction of the system resilience is always below the 10.7%. Furthermore, the solution which minimizes this objective provides a Resilience Deviation Index equal to zero. This solution corresponds to the non-sectorization of the network, thus highlighting that in the proposed methodology the non-intervention scenario is evaluated as well as the partitioning strategies.



**Fig. 4.13 – Optimal solutions for the sectorization of the 25-nodes network**

A detailed description of the eight optimal designs of the network sectorization is provided in Fig. 4.13. The solutions are labelled for increasing values of the Total Cost Function (i.e. Solution 01 is the one which minimizes the TCF, Solution 02 is the second best, etc.). The diversity achieved in the optimal set is highlighted by the significant differences that can be observed between the obtained designs.

Nevertheless, it is important to point out that in some cases (Solutions 02 and 07) the design of the DMAs does not seem satisfactory because some of them consist of just one or two nodes. However, this outcome can be ascribed to the very reduced size of the network, in which significant shares of the total demand are concentrated in single nodes, thus ensuring the compliance with the problem constraint relevant to the size of the DMAs.

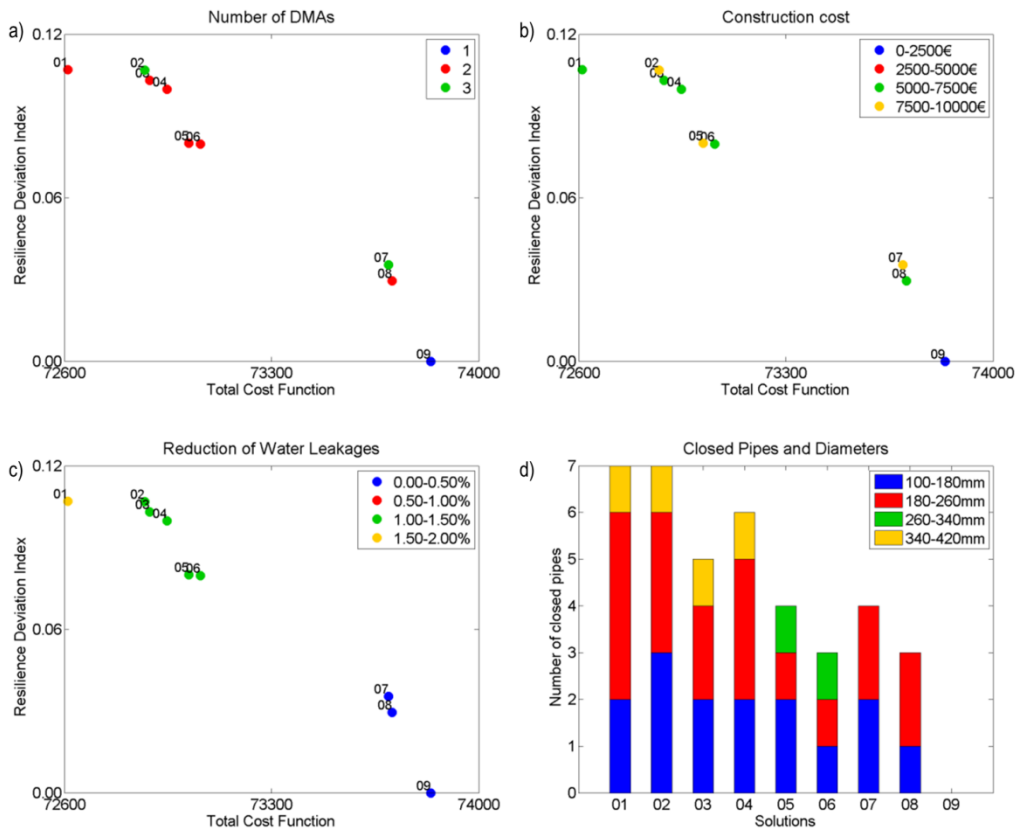


Fig. 4.14 – 25-nodes network: technical features of the obtained solutions

In the worst case the minimum water pressure at the most critical node in the network is 30.1 m. Moreover, according to what is stated in Sec. 4.14, further plots are reported in Fig. 4.14 in order to provide a better understanding of the obtained solutions under a technical point of view. For example, Fig. 4.14a clearly shows that having two DMAs is the best choice. Indeed, the upper bound of this quantity is only reached in the two solutions that have been discussed previously. Furthermore, in this case there is no evidence of a clear relationship between the number of DMAs and the values of the objective functions.

The same happens for the construction cost (Fig. 4.14b), which values range from 5,760 (Solution 03) to 9,360 € (Solution 02). However, in most of the cases it ends up being quite far from the assumed budget limit. Conversely, the reduction of the water leakages (Fig. 4.14c) shows an evident trend which depends on both the objectives. In particular, it increases with the Resilience Deviation Index and it decreases with the Total Cost. Therefore, the estimated savings in terms of water losses (between the 0.30% in Solution 08 and the 1.81% in Solution 01) are shown to have great influence on the operational cost of the network. At the same time, the hydraulic reliability of the WDN is negatively affected by the reduction of the water leakages, since they are opponents in the dependence from the water pressures.

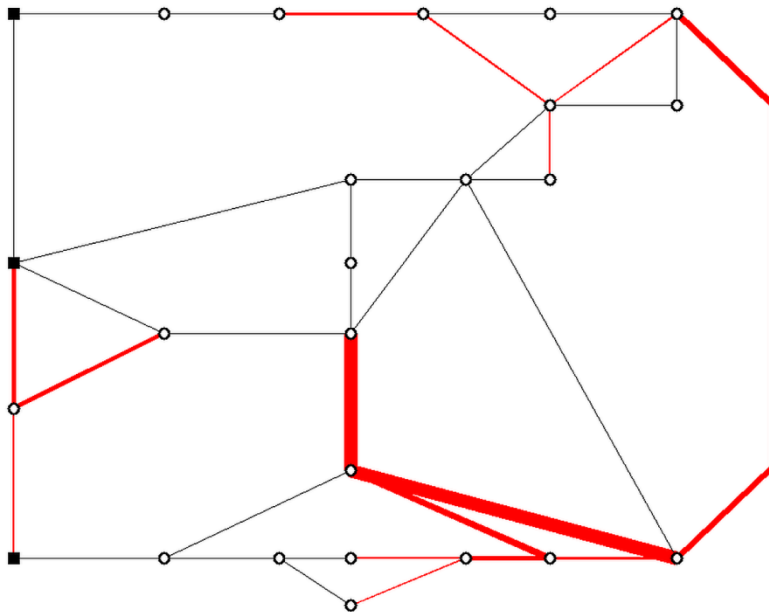
As regards the intercepted pipes, Fig. 4.14d shows that the obtained solutions involve, in the worst case, the closure of 7 links in the network. Furthermore, they are mainly characterized by a small pipe diameter, as specified in the criteria adopted in best practice. In addition, as the Total Cost is predominantly ruled by the water leakages, its minimization requires the closure of a larger number of pipes (Solutions 01 and 02).

At this stage a further specification should be provided about the construction cost for the latter solutions. Despite having at least one diameter smaller than those in Solution 01, Solution 02 requires a greater expenditure in the short-term, even though they both involve the same amount of intercepted pipes. This happens because Solution 02 consists of 3 DMAs (instead of 2), one



of which is supplied in cascade. The increased cost is due to the flow-meter that has to be installed on the entry connection of the isolated DMA.

Finally, in Fig. 4.15 the “closure frequency” of the pipes is shown. The black lines represent the links that are never closed in any solution. The red lines correspond to the pipes intercepted in at least one solution, and their thickness varies according to the number of solutions in which they are assumed as closed. Pipes 20 and 21 (see Fig. 4.9) have the highest closure frequency, and are followed by pipes 19 and 35.



*Fig. 4.15 – 25-nodes network: closure frequency of pipes*

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## **A real case study: the water distribution network of Pianura**

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In this chapter the application to a real-world case study is presented, in order to provide evidence of the potential of the proposed methodology for the automatic sectorization of the water distribution networks.

The analysis is performed according to the general approach and assumptions already adopted for the example described in the previous chapter. However, in this case, a more in-depth description of the network is provided, which is based on the information provided by the WDN manager (Acqua Bene Comune ABC Azienda Speciale Napoli). The obtained results are discussed in detail, as they yield realistic indications for the actual implementation of the network sectorization project.

### **5.1. The water distribution network of Pianura**

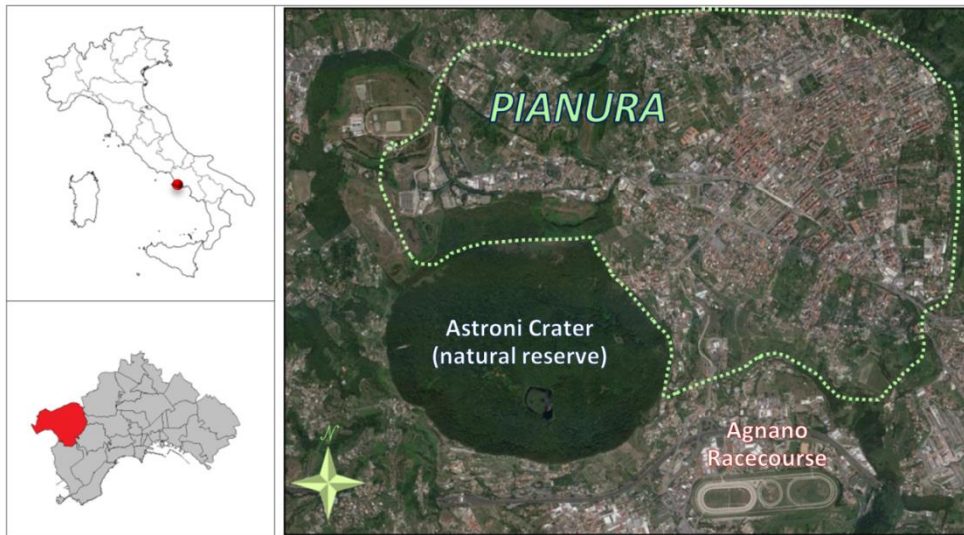
Pianura is a western suburb of the city of Naples, which is the administrative centre of the southern Italian region Campania. This district stretches just south of the Camaldoli hill up to the close “Montagna Spaccata”, and was an independent municipality until 1926. Today it is surrounded by five other Neapolitan neighbourhoods (Chiaiano, Arenella, Soccavo, Fuorigrotta and Agnano) and the municipalities of Quarto and Marano di Napoli (Fig. 5.1).

Pianura covers an overall area of 11.45 km<sup>2</sup> and has a population of 58,000 inhabitants (last survey, dated 2001). However, the water distribution system addressed in this study only supplies a portion of this neighbourhood consisting

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of approximately 40,000 people, to whom a per-capita demand of 250 litres/inhabitant/day is provided.

At present, the network is supplied by a unique water source, namely the *S. Giacomo* tank (Fig. 5.2a), which presents a maximum level of 230 m AMSL and a capacity of 60,000 m<sup>3</sup>. The ground elevations vary greatly, from a minimum value of 48 m AMSL in the southern area to a maximum of 180 m AMSL at its central-northern boundary (Fig. 5.2b).



*Fig. 5.1 – Location of the study area (Pianura, Naples, Italy)*

As shown in Fig. 5.2a, the system is characterized by a looped structure with a high redundancy, especially in most densely populated area (at north). Conversely, in the outskirts the houses are supplied with single branches, some of which having a remarkable length.

The main trunk (red line in Fig. 5.2a) consists of a DN600 steel pipe linking the tank to the eastern end section of the main road in the area (Montagna Spaccata road); the same pipe reaches the opposite end of the road with a reduction of the diameter (from DN600 to DN500) at its middle section.

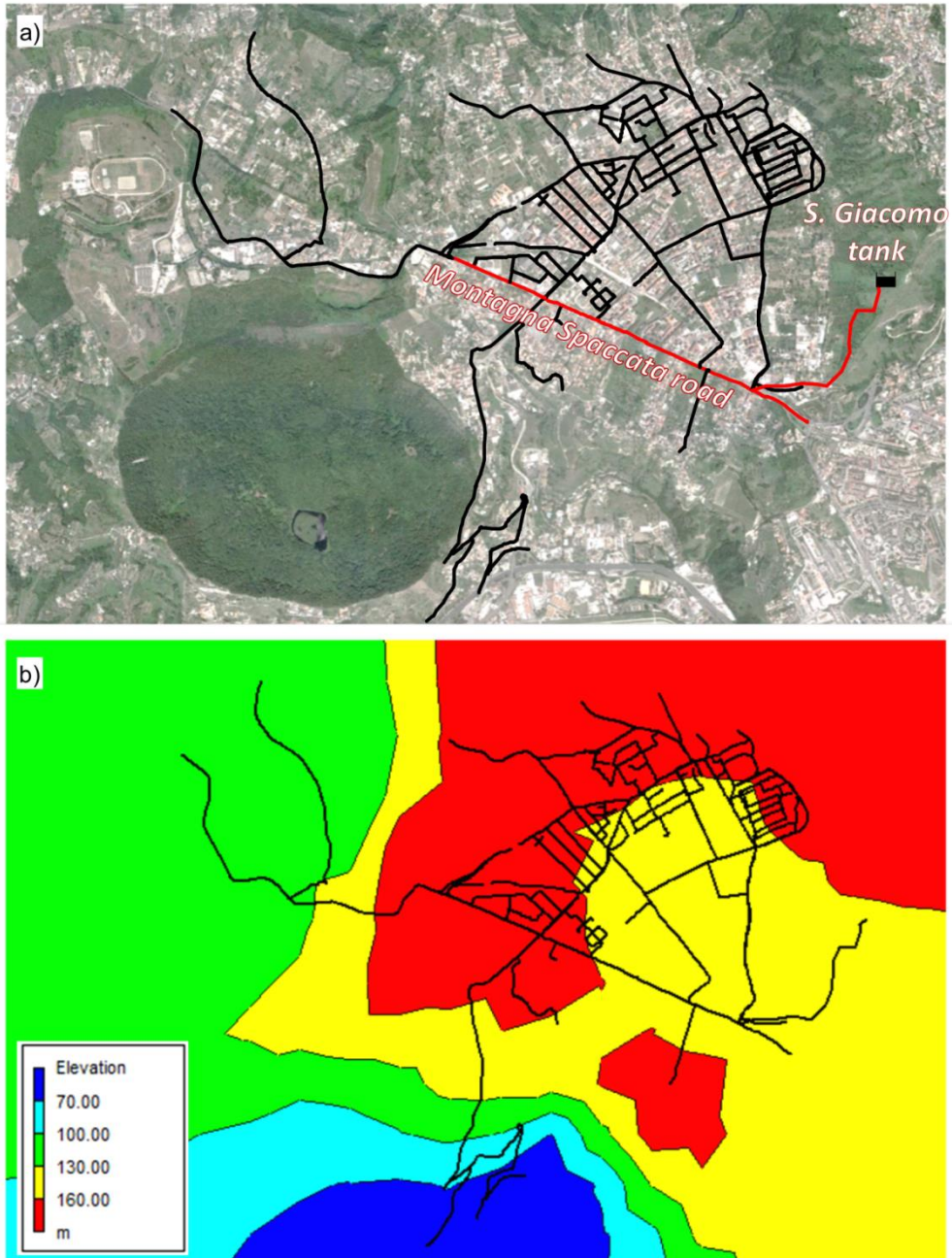


Fig. 5.2 – Water distribution network of Pianura: overview and main trunk (a); ground elevations (b)

The other network pipes have diameters ranging in the interval between 40 and 600 mm, and they are predominantly made of steel and grey and cast iron. A detailed description of the distribution of the pipe diameters according to the respective lengths is reported in Fig. 5.3.

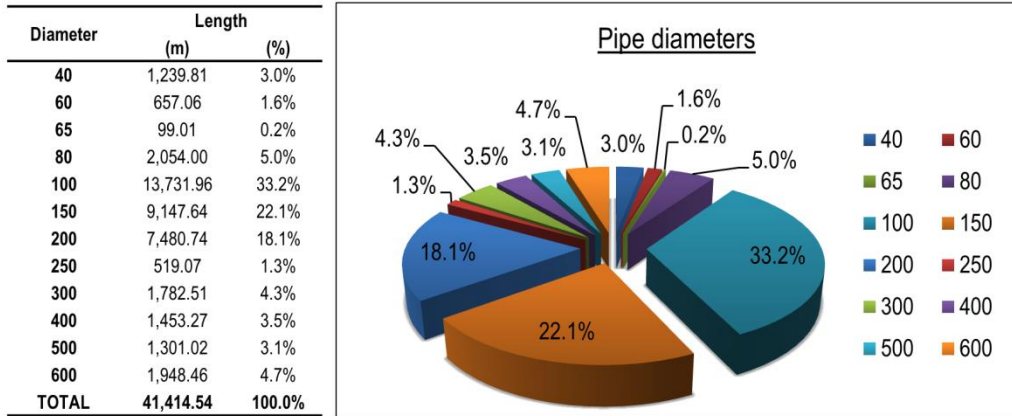


Fig. 5.3 – Lengths of the network pipes classified by diameter (in mm)

The characterization of the hydraulic behaviour of the network is performed on the basis of the measurements performed by the network manager. For example, Fig. 5.4 shows the discharges and the water pressures detected during the period between January 27 and February 5, 2010, at the entry point of the network (namely the section of the main trunk at downstream of the tank, just a few meters before the intersection with Montagna Spaccata road).

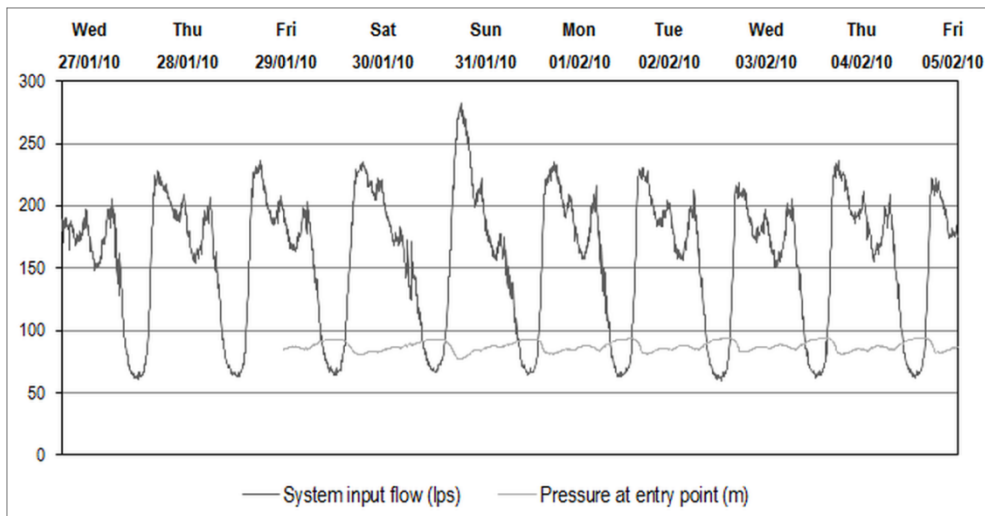


Fig. 5.4 – Flow and pressure measurements at the entry point of the network

The available information makes it possible to highlight the following features of the hydraulic operation of the network:



- the daily trend of the system input flow during weekdays does not show significant variations over time, and it presents a strong morning peak (around 8-9 am) with a maximum value of 230-240 lps. Two other peaks of lesser magnitude can be observed later in the day, respectively at 2 and 8 pm. Furthermore, a nearly constant minimum flow of approximately 60-70 lps is supplied during the night hours;
- the same trend presents significant differences during weekends. In particular, the morning peak is shifted at 9-10 am on Saturdays and even later on Sundays (10-11 am). In addition, the afternoon and evening peaks cannot be observed so clearly during these days. Nevertheless, the Sunday morning peak shows a maximum value significantly larger than those observed during weekdays (about 260-280 lps);
- the hourly consumption coefficient (i.e. the ratio between the hourly flow and the daily average) presents rather small variations during the day, with maximum values around 1.50 (1.80 on Sundays);
- the trend of the water pressure at the entry point of the network shows a very reduced oscillation, with amplitude in the order of 1 bar (between 82 and 93 m). According to the previous remarks, the minimum value (approximately 77 m) is observed during the Sunday morning peak. The same occurs at other different locations in the network.

Further information can be deduced by analysing the billed consumption, whose values in the three years period between 2007 and 2009 are reported in Tab. 5.1. The mean value of the consumption (3,531,300 m<sup>3</sup>/year, equivalent to 111.9 lps) is consistent with the above mentioned demand per-capita. Also in this case, the detected fluctuations are quite modest, thus confirming the very stable operating conditions of the network in terms of both system input flow and water pressures.

Year	Total billed consumption	
	(m <sup>3</sup> )	(lps)
2007	3,506,000	111.2
2008	3,610,000	114.2
2009	3,478,000	110.3
<b>Mean</b>	<b>3,531,300</b>	<b>111.9</b>

*Tab. 5.1 – Total billed consumption for Pianura (years 2007-2009)*

According to the retrieved data, it is then possible to perform an estimate of the water balance in order to assess the amount of the water losses in the network. In particular, the average daily flow supplied during the observation period is equal to 157 lps (4,951,000 m<sup>3</sup>/year).

Nevertheless, a more in-depth analysis of the distribution systems showed that part of this flow is conveyed outside of the network towards the nearby municipality of Pozzuoli. This discharge amounts to approximately 20 lps (631,000 m<sup>3</sup>/year). As a result, the actual value of the supplied volume is 4,320,000 m<sup>3</sup>/year; considering the 3,531,300 m<sup>3</sup>/year of billed consumption, an estimate of 789,000 m<sup>3</sup>/year (25 lps) of lost water is given.

Therefore, the total losses (apparent and real) in the water system of Pianura account for about the 18% of the total supplied volume. Assuming a general value of 30% for the apparent losses, it can be concluded that the physical water losses amount to approximately 592,000 m<sup>3</sup>/year (17.5 lps, i.e. the 13.7% of the system input flow).

In order to perform the most accurate localization of the water leakages in the network, the maintenance reports provided by the system manager can be assessed to pinpoint the vulnerability of the different network areas. In particular, the analysis of the interventions operated during the year 2009 and the first trimester of 2010 makes it possible to identify the most critical sites, which are mainly located in the northern area of the network. This results in an increased accuracy in the quantification and in the localization of the emitter nodes for modelling the pressure-dependent water leakages.

## 5.2. Hydraulic modelling of the network

The available information makes it possible to build the hydraulic model of the network that is required for running the simulations. To this aim, the data collected from the GIS (*Geographical Information System*) provided by the network manager are used to outline the network graph and the features of its main components.

The hydraulic modelling of the WDNs is usually performed by applying a *skeletonization* procedure, which mainly consists of the following steps: i) removals and unifications; ii) adoption of equivalent pipes. The first category includes some operations that allow simplifications without involving any modification of the hydraulic features of the pipes (length, diameter, roughness). These operations are namely the merging of pipes with similar features and reduced length, as well as the removal of short dead-end trunks (Fig. 5.5a) and micro-networks with single access (Fig. 5.5b).

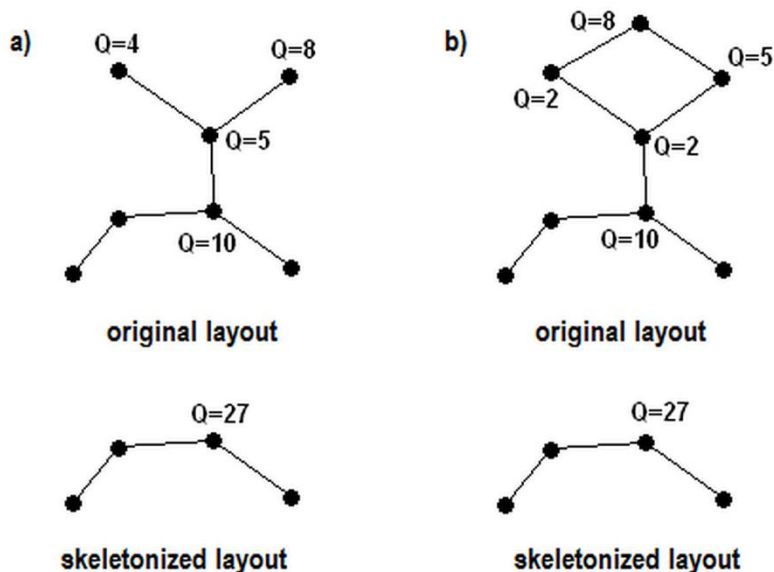
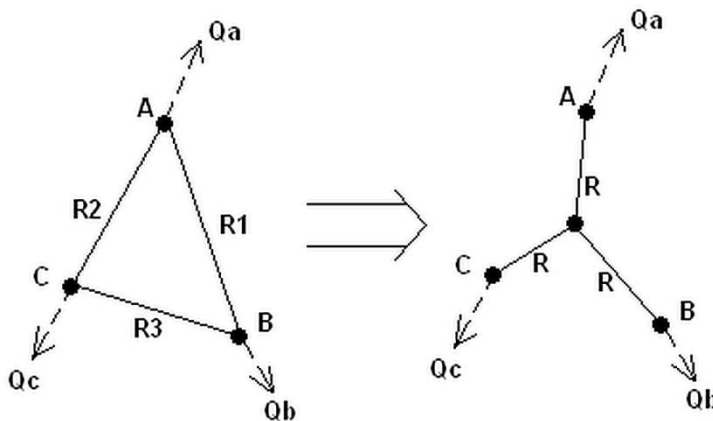


Fig. 5.5 – Examples of pipe removals: (a) dead-trunks; (b) micro-networks with single access (Portolano 2008)

The use of equivalent pipes involves the substitution of some network structures with other ones having a completely different layout but a similar hydraulic behaviour (obtained through mathematical relationships). In this category we can find: i) the replacement of parallel pipes with an equivalent one; ii) the  $\Delta$ -Y transform (*delta-wye transform*, widely used in the analysis of three-phase electric power circuits), consisting of the substitution of a loop with a structure made of 3 pipes connected to a common node (Fig. 5.6); iii) the opening of network loops through the removal of the pipes characterized by very low hydraulic conductivity.

It should be remarked that only the latter procedure allows a significant reduction of the model dimension. However, since this operation involves a drastic change in the network layout, particular attention should be paid in its application because the obtained results can significantly differ from the originals.



*Fig. 5.6 – Delta-wye transform (Portolano 2008)*

However, the skeletonization of the network also involves a reduced capacity of handling the information and the results obtained from the analysis. Furthermore, in this case, the implementation of the input data is facilitated by the computer database containing the information about the network topology, and the adopted hydraulic simulator ensures a good computational performance.

Therefore, a marked simplification of the layout is not as necessary, and the hydraulic model of the WDN of Pianura is obtained through a skeletonization that is only based on removals and unifications. Its final configuration consists of 269 nodes (268 junctions and 1 reservoir), 313 links (i.e. pipes), and a total number of 45 loops.

The Manning roughness is assumed for each pipe for the calculation of the head-losses with Chezy's formula. The material and the age of the pipes are taken into account for the assignment of the coefficients. As for the water demands, their variability in time and space is evaluated according to the information described in the previous section.

First of all, the average values ( $Q_{i,k}$ ) of the supplied flows (i.e. customer water demands plus leakages) for every day ( $k$ ) and hour ( $t$ ) in the sampling period are computed as follows:

$$Q_{t,k} = \frac{\sum_{j=1}^N Q_{j,t,k}}{N} \quad (5.1)$$

where  $Q_{j,t,k}$  and  $N$  are, respectively, the  $j$ -th flow sample and the total number of samples detected during the  $i$ -th hour of the  $k$ -th day.

It must be noted that, even though different consumptions have been observed in weekdays and Sundays, in this case only the ordinary operation of the network is considered (i.e. the operation in weekdays). Furthermore, since the measurements were performed during the first two months of the year, the obtained values are obviously relevant to the winter time. However, the evaluation of the seasonality of the water demand lies outside the purpose of the present work and it is not taken into account here.

That said, with reference to the quantities in Eq. 5.1, the corresponding values of the hourly average flows ( $Q_t$ ) are evaluated:

$$Q_t = \frac{\sum_{k=1}^7 Q_{t,k}}{7} \quad (5.2)$$

The same can be done for the detected water pressures (with similar notation):

$$P_{t,k} = \frac{\sum_{j=1}^N P_{j,t,k}}{N} \quad (5.3)$$
$$P_t = \frac{\sum_{k=1}^7 P_{t,k}}{7}$$

Next, the average daily demand ( $Q_m$ ) is calculated as follows:

$$Q_m = \frac{\sum_{t=1}^{24} Q_t}{24} \quad (5.4)$$

The quantities in Eqs. 5.2 and 5.4 can be used for calibrating the time pattern of the water demands of customers. Each hourly multiplier ( $\chi_t$ ) is provided by the ratio between the hourly water demand and its daily average value:

$$\chi_t = \frac{Q_t}{Q_m} \quad (5.5)$$

The validity of this assumption can be ascribed to the relatively low amount of real water losses in the network (see Sec. 5.1). In different cases, when the water leakages represent a significant rate of the total water demand, the time pattern of the latter cannot be used for modelling the hourly consumption of customers. However, the obtained results for the WDN of Pianura are reported in Fig. 5.7.

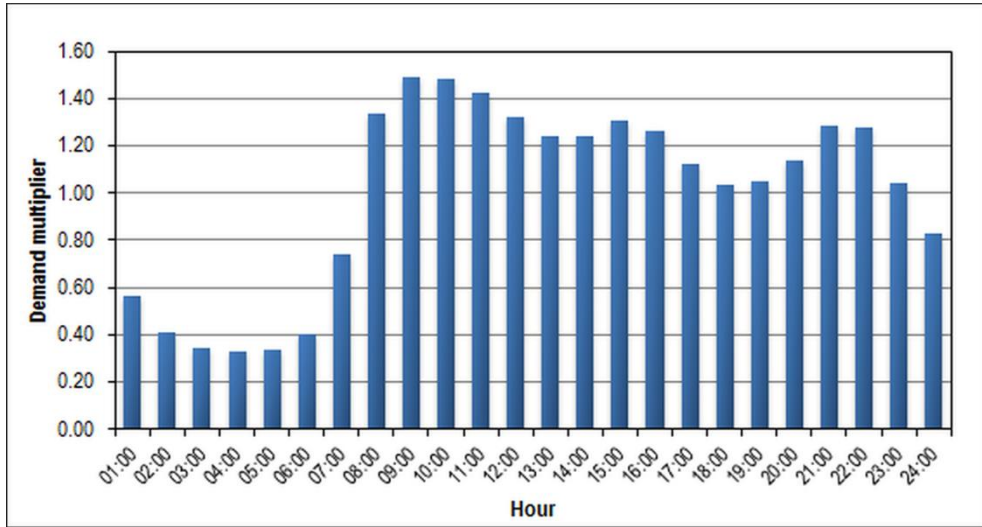


Fig. 5.7 – Daily time pattern of the water demands of customers

The actual consumption of the customers is evaluated according to the billed water volumes ( $V_h$ ) registered by a number of flow-meters at different locations in the network. The average demand at the  $h$ -th flow meter ( $Q_{c,h}$ ) is obtained by dividing the billed volume by the number of days in the sampling period ( $N_{days}$ ) and the number of seconds in one day:

$$Q_{c,h} = \frac{V_h}{86,400 \cdot N_{days}} \quad (5.6)$$

The total billed consumption is given by the sum of the detected quantities over all the considered flow-meters (in number of  $N_h$ ):

$$Q_c = \sum_{h=1}^{N_h} Q_{c,h} \quad (5.7)$$

As stated in the previous section, in this work the total water losses are assumed to be distributed with 70% in water leakages and the remaining 30% in apparent losses. The real value of the average daily water demand (namely the *base demand*,  $BD$ ) is then provided by the following:

$$BD = Q_c + 0.3 \cdot (Q_m - Q_c) \quad (5.8)$$

The distribution of this quantity among the demand nodes in the network requires a detailed study about the influence area assigned to each of them. To this aim, the type and the number of buildings resting on every node, as well as the length of the pipes connected to nodes themselves are taken into account.

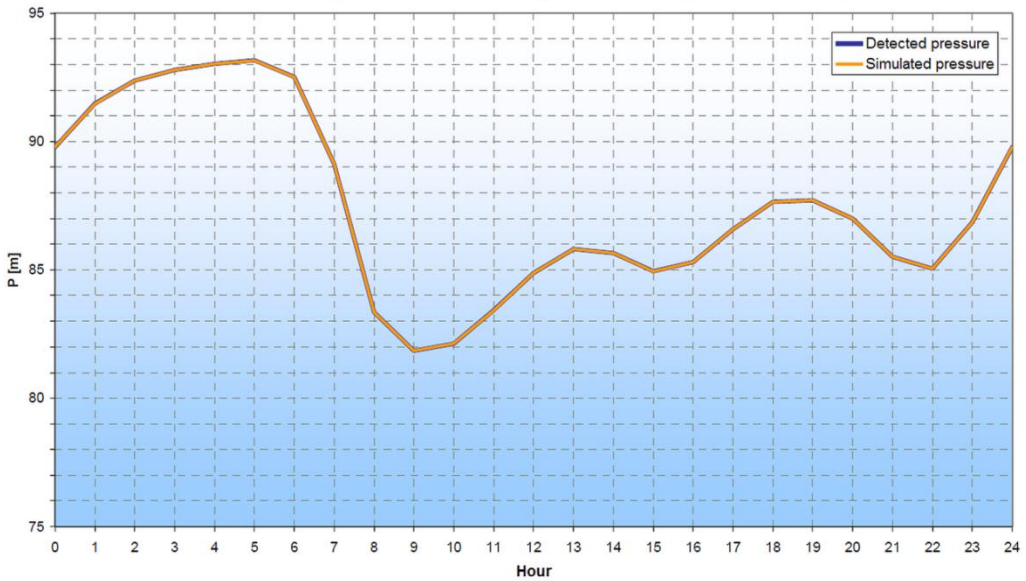
As for the real losses, pressure-dependent leakages are modelled according to the monomial formulation (Lambert 2001) with two parameters. In particular, the *emitter exponent* ( $\beta$ ) of every emitter node is set equal to the standard value of 0.5 (from Torricelli's equation), while the *discharge coefficient* ( $\alpha$ ) is calibrated on the basis of the previously obtained quantities. First, the water demands are assigned to each node ( $i$ ) according to the node type:

$$q_{i,t} = \begin{cases} \chi_t \cdot BD_i & \text{if } i \text{ is demand node} \\ \chi_t \cdot BD_i + \alpha P_{i,t} & \text{if } i \text{ is demand node + emitter} \\ \alpha P_{i,t} & \text{if } i \text{ is emitter} \end{cases} \quad (5.9)$$

where  $BD_i$  and  $P_{i,t}$  are, respectively, the base demand and the water pressure at time  $t$  relevant to the  $i$ -th node.

Next, the time pattern of the water pressure at the entry point of the network is considered. The calibration of the discharge coefficient is then performed by running the hydraulic simulation of the network several times with different values of  $\alpha$  until the same pattern is reproduced. As shown in Fig. 5.8, the obtained value of  $\alpha = 0.023$  provides a nearly perfect overlap between the detected water pressures and the simulated values.





*Fig. 5.8 – Calibration of the discharge coefficient. Comparison between the detected and the simulated pressures*

The so built hydraulic model allows a preliminary evaluation of the operating conditions of the network. In particular, no criticalities can be found about the water supply, which is able to meet the demand of customers with appropriate pressures also during the peak hour (Fig. 5.9a).

Furthermore, the oscillation of water pressure for a large part of the network nodes is contained in a rather narrow range. However, although the minimum value (28.20 m) is consistent with the service requirements (20 m), very huge pressure heads (over 100 m) can be observed in the areas served by the long dead-end branches, especially during the night hours (Fig. 5.9b).

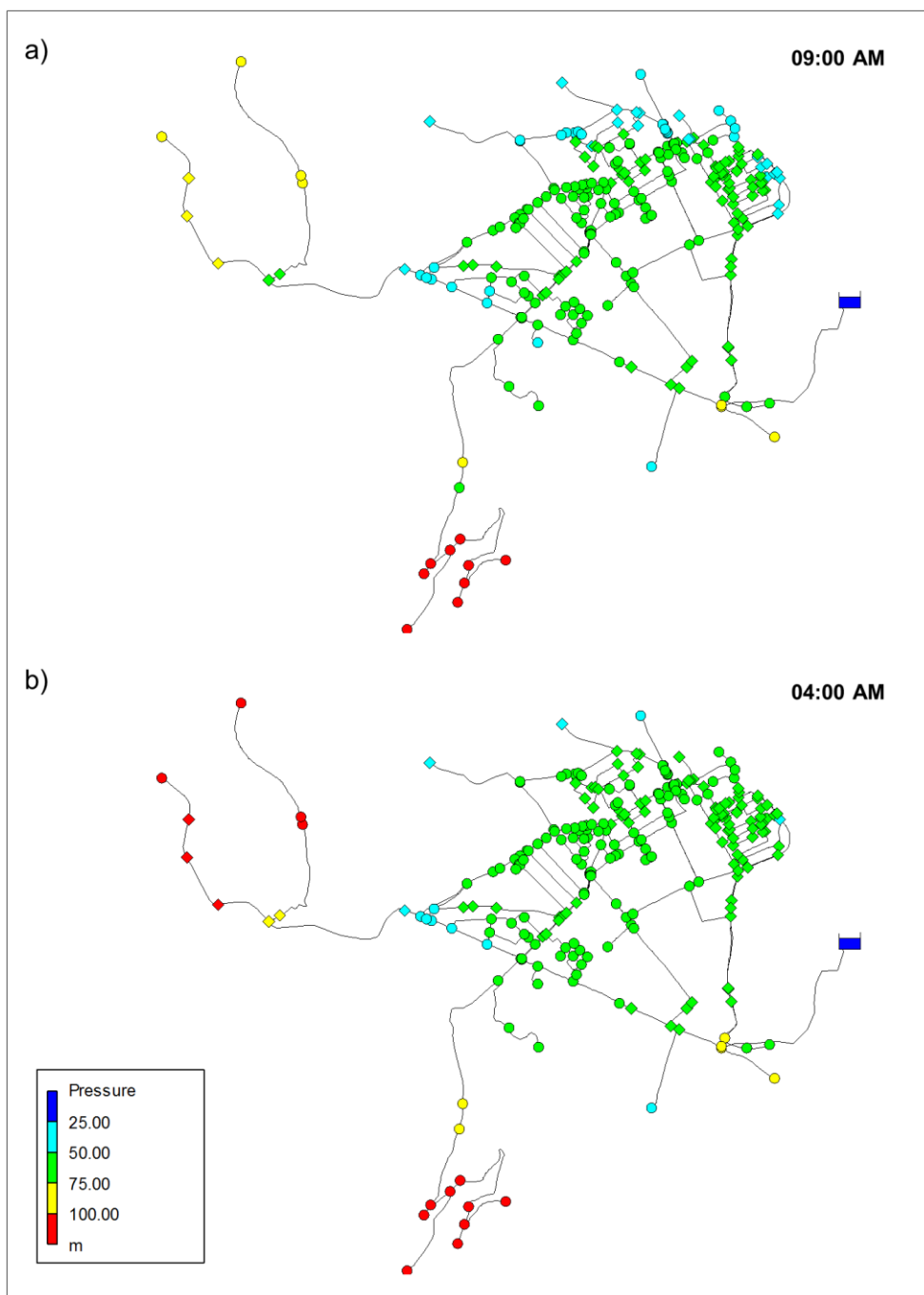


Fig. 5.9 – Water pressures in the starting configuration of the network. Peak hour (a); minimum consumption (b)

### 5.3. Optimal design of DMAs

The methodology described in the Chapter 4 is here applied for designing the optimal sectorization of the water distribution network of Pianura. To this aim, the main features of the network are firstly summarized in Tab. 5.2.

Number of demand nodes	Number of pipes	Number of reservoirs	Total number of customer connections	Minimum required pressure at demand nodes (m)
268	313	1	17,000	20.00

*Tab. 5.2 – Sectorization of the WDN of Pianura: network features*

In this case, the number of pipes and nodes is about 10 times larger than that of the example discussed in the previous chapter. Consequently, the complexity of the problem and the required computational time are significantly increased, and therefore the present application can be considered as a good test for the evaluation of the performance of the developed algorithm.

#### 5.3.1. Problem setup

The analysis is performed by considering the standard values for the minimum and the maximum number of customer connections per each DMA (500 and 5,000, respectively). The choice of the upper bound for the number of DMAs is made with the aim of ensuring a good balance between the broadest possible investigation of the problem and the technical feasibility of the provided solutions. According to the size of the network, this parameter is here set equal to 5.

Also in this case, the optimization is performed with the aim of reducing the total operational cost of the network as a whole. For this reason, equal weights (0.50) are assigned to the required investment and to the cost due to the water leakages, so that the TCF is half the annual operational cost. A budget limit of 50,000 € is introduced, as well.

Min. number of connections per DMA ( <i>MinCon</i> )	Max. number of connections per DMA ( <i>MaxCon</i> )	Maximum number of DMAs ( <i>NDMA<sub>max</sub></i> )	Budget limit for the required investment ( <i>CB<sub>max</sub></i> )	Weight for the required investment ( <i>w<sub>B</sub></i> )	Weight for the cost of water leakages ( <i>w<sub>L</sub></i> )
500	5,000	5	50,000 €	0.50	0.50

Tab. 5.3 – Sectorization of the WDN of Pianura: analysis options

The selection of the unit costs (Tab 5.4) of the shut-off valves and of the flow-meters is performed on the basis of a brief market survey. According to a service life of 50-60 years, a 5% annual discounting rate is adopted. The estimated cost of the water leakages is 0.18 €/m<sup>3</sup>, and it takes into account both the operational cost and the environmental effects. It is also consistent with the wholesale price of purchasing water indicated in the guidelines provided by the local authority (Regione Campania 2013).

Pipe diameter (mm)	Cost of shut-off valve (€)	Cost of flow-meter (€)
40	120.00	1,665.00
60	135.00	1,675.00
65	160.00	1,700.00
80	175.00	1,730.00
100	215.00	1,770.00
150	340.00	1,950.00
200	560.00	2,245.00
250	845.00	2,430.00
300	1,315.00	2,860.00
400	3,755.00	5,345.00
500	8,070.00	6,470.00
600	9,665.00	7,950.00
<b>Cost of water leakages (€/m<sup>3</sup>)</b>		<b>0.18</b>
<b>Annual discounting rate</b>		<b>0.05</b>

Tab. 5.4 – Sectorization of the WDN of Pianura: unit costs

### 5.3.2. Test cases

The application of the optimization algorithm to the water distribution network of Pianura also requires the specification of the parameters of the NSGA-II. In this regard, a preliminary analysis is performed by using the best combination of values that was found in the calibration described in Sec. 4.5.1. This first test case is analysed through 5 different runs with randomly generated initial populations. The number of planned evaluations is 25,000 (Tab. 5.5)

Population size	Number of generations	Distr. index for crossover	Distr. index for mutation	Number of runs	Number of evaluations
50	100	1	20	5	25,000

*Tab. 5.5 - Sectorization of the WDN of Pianura: parameters of the NSGA-II (first test case)*

However, in order to assess the convergence of the obtained results towards the global optimum, a further test case is considered, which consists of a very large number of evaluations. Hence, a single run of the optimization algorithm is performed assuming that the population size is equal to 100 and that the number of generations is 1,000. The resulting number of evaluations is four times larger than the one adopted in the first test case (Tab. 5.6).

Population size	Number of generations	Distr. index for crossover	Distr. index for mutation	Number of runs	Number of evaluations
100	1,000	1	20	1	100,000

*Tab. 5.6 - Sectorization of the WDN of Pianura: parameters of the NSGA-II (second test case)*

### 5.3.3. Results and discussion

The analysis was run using the same machine described in Sec. 4.5.3. The first series of tests was finished in approximately 5 hours and 42 minutes, while the second required over 17 hours and 30 minutes to be completed.

In order to measure the computational complexity, it can be highlighted that the time taken for running 125,000 evaluations on this network is comparable to the time required by the complete analysis (3,429,000 evaluations) on the example network by Jowitt and Xu. However, this could be considered as a good result, especially if the great complexity of the addressed problem is taken into account.

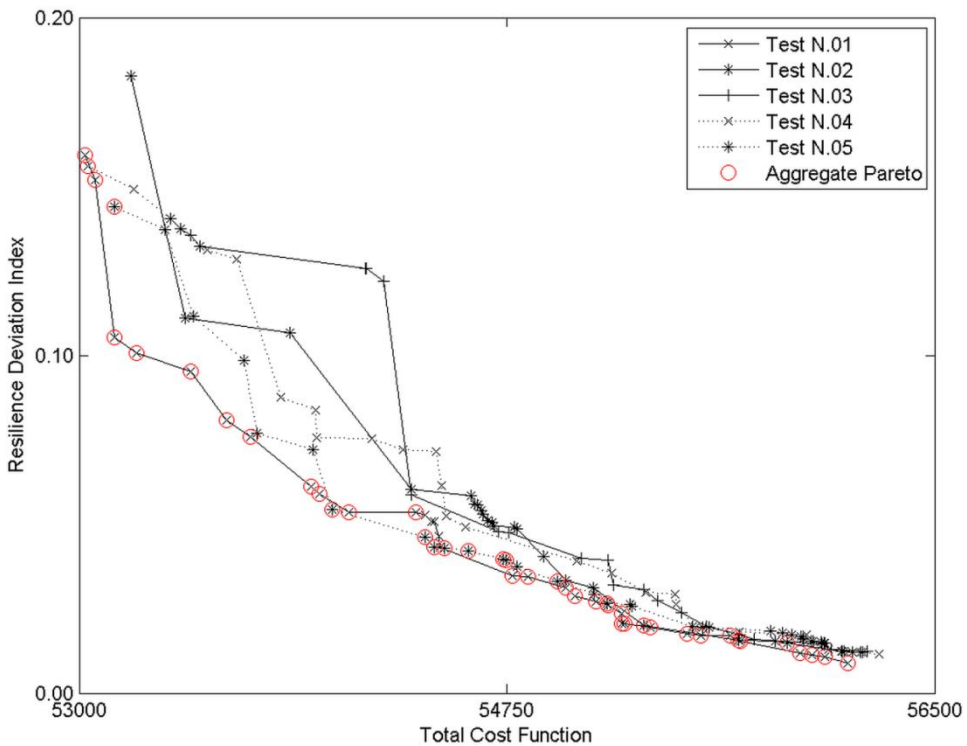


Fig. 5.10 – Sectorization of the WDN of Pianura: aggregation of the Pareto fronts (first test case)

The outcomes of the tests described in Tab. 5.5 (namely those with reduced numbers of evaluations) are presented first. Figure 5.10 shows the comparison between the Pareto fronts obtained in the 5 runs of the designed algorithm. It can

clearly be observed that there is a good overlap of the solutions which minimize the Resilience Deviation Index, while more spread is detected in the optimization of the Total Cost Function.

Further evidence about this feature can be provided by plotting the *attainment surfaces* (Fonseca and Fleming 1996) relevant to different percentiles. The Attainment Surface (AS) is namely the boundary line that divides the solutions in the objectives space that are dominated by the optimal set resulting from the multi-objective algorithm from those that are not. In more detail, the  $k\%$ -AS is the boundary line of the portion of the objectives space that is attained in  $k\%$  of the runs of the optimization algorithm.

Figure 5.11a shows the *best*-, the *median*- and the *worst*-AS, which are namely the attainment surfaces corresponding to the solutions that are dominated in 0% (never), 50% and 100% (always) of the runs. The very narrow stripe defined by the best-AS and the worst-AS at low values of the Resilience Deviation index indicate the good performance of the algorithm in the optimization of this objective.

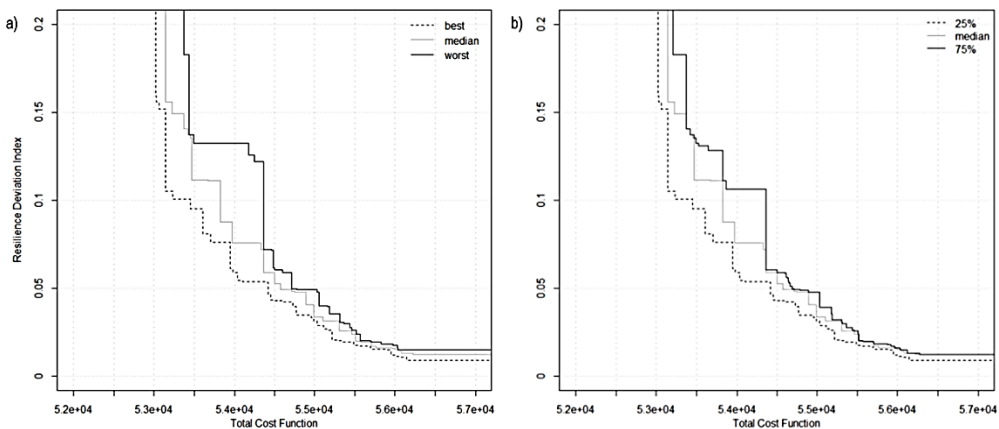


Fig. 5.11 – Attainment surfaces at different percentiles

Conversely, less convergence is observed in the minimization of the TCF, while the worst performance is obtained in the intermediate area, where the best trade-off solutions are located. A more clear perspective is provided by the analysis of the 25th- and 75th-AS (Fig. 5.11b), that are quite close to the medium-AS in the

extreme regions, while they stay rather far from each other in the intermediate one.

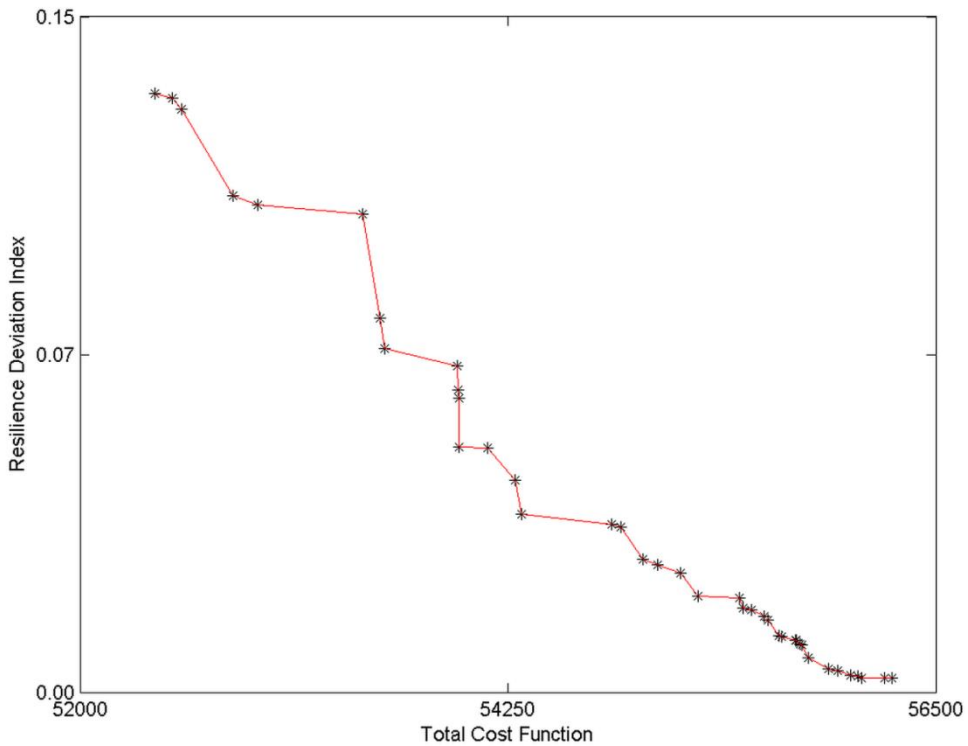


Fig. 5.12 – Pareto front obtained in the second test case

The second test case (100,000 evaluations) was run just once, and therefore it can be directly described through the analysis of the Pareto front obtained at the end of the optimization process (Fig. 5.12). From a qualitative point of view, it can be noted that the area corresponding to the minimization of the Resilience Deviation index is covered in a similar way to the one observed in the first set of tests. This happens in terms of both values of the objective functions and “density” of the obtained solutions (i.e. the number of solutions in the same area).

On the contrary, a different performance is detected for what concerns the minimization of the TCF. For example, the best solution in this regard is characterized by values of 52,394 and of 0.1329 for the TCF and the Resilience Deviation Index, respectively. Conversely, in all of the runs performed in the

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previous case the TCF never falls below 53,000 (53,026 is the best), and the corresponding values of the Resilience Deviation Index are generally higher (between 0.10 and 0.15).

However, a more in-depth comparison of the outcomes is required, in which the whole performance of the two tested algorithms (i.e. the two algorithms characterized by different parameters) is taken into account. To this aim, instead of using the hypervolume, a different tool is adopted, namely the *first-order Empirical Attainment Function* (here referred as EAF).

The EAF provides an estimate (performed according to the obtained results) of the true and unknown Attainment Function, that is a function defined in the domain of the objectives and having values in the range [0;1]. Each of these values expresses the probability with which the corresponding solution is attained (i.e. dominated) by a random optimal set obtained through the optimization algorithm (Grunert da Fonseca et al. 2001). In the two-dimensional space (bi-objective optimization), the plots of the EAF and of the attainment surfaces allow a powerful side-by-side comparison of the outcomes, thus providing a very effective assessment of the performance of the algorithms in a graphical way.

Figure 5.13 shows the plots of EAFs of the two tested algorithms that are obtained through the “eaf” package of the statistical software R (López-Ibáñez et al. 2010). In particular, the intensity of the shaded area increases with the value of the EAF. The dashed line in the left plot represents the medium-AS obtained for the first algorithm (i.e. the first test case). Furthermore, the *grand best*- and the *grand worst*- Attainment Surfaces are shown, which consist of the best and of the worst sets of points attained over all the runs of both the algorithms (so they are identical for the compared plots). Obviously, the single run performed in the second test case partially affects the appearance of the corresponding plot (on the right side of Fig. 5.13), in which the dashed line is not displayed and there are not intermediate values of the EAF between 0 and 1.

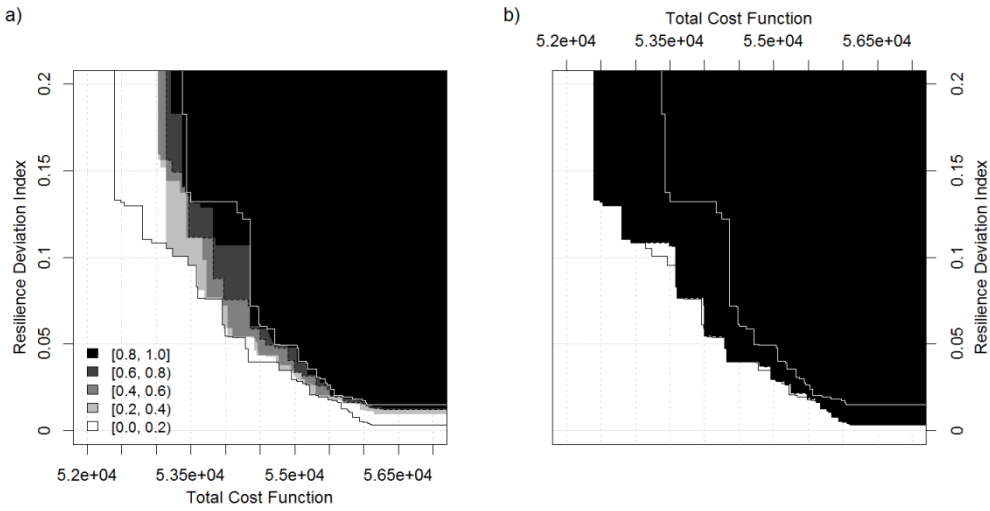


Fig. 5.13 – Comparison between the EAFs: first (a) and second (b) test cases

The comparison provides further evidence about the best performance obtained in the second test case: indeed, the values of the EAF in the stripe defined by the grand best- and the grand worst-AS are substantially always higher than those observed in the first case. This is confirmed by the plots in Fig. 5.14, which show the differences between the EAFs. In both the plots, the differences in favor of the corresponding algorithms are shown (first algorithm on the left, second on the right). In this case, the intensity of the shaded area encodes the magnitude of the observed difference.

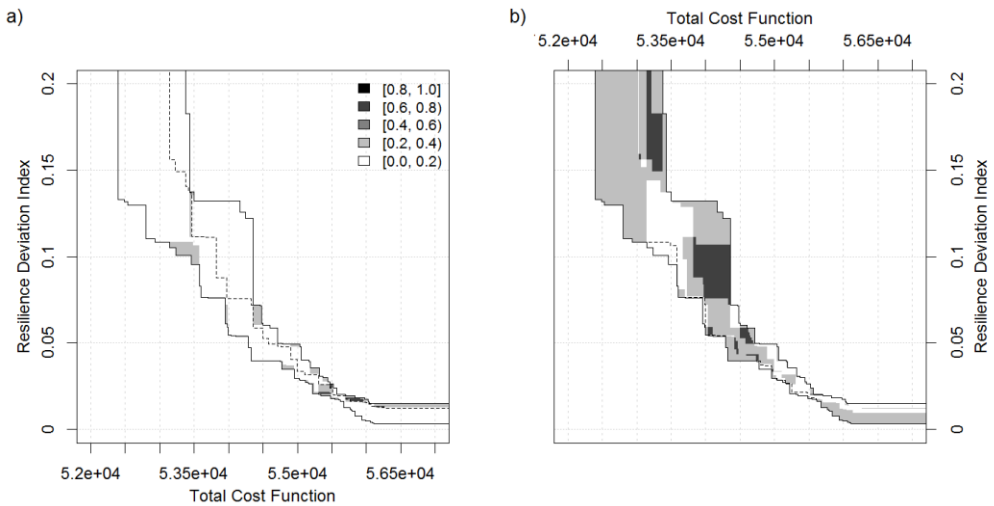


Fig. 5.14 – Differences between the EAFs: in favor of the first (a) and of the second (b) algorithm

Therefore, in this case the optimization algorithm with a larger number of evaluations exhibits a better performance throughout the Pareto front, except for a little portion in the intermediate area (where the solutions show more spread). This interesting and expected outcome is consistent with the significantly larger size of the real-world network compared to that of the example network analyzed in the previous chapter.

However, in order to proceed to the technical description of the results, the best results are collected through the aggregation of the obtained Pareto fronts, as explained in Sec. 4.5.2. The resulting optimal set includes 52 different solutions, whose main features are summarized in the plots of Fig. 5.15.

What is particularly notable is that in this case the non-intervention scenario (i.e. the solution with Resilience Deviation Index equal to 0) is not included in the Aggregate Pareto front, thus supporting the remarkable opportunities offered by the sectorization for the actual improvement of the network operation. Furthermore, in the worst case the reduction of the system reliability is quantified in 13.29% (Resilience Deviation Index equal to 0.1329).

As shown in Fig. 5.15a, almost all the solutions (except one) consist of 4 or 5 DMAs. Even though also in this case there is not a clear relationship with the objectives, it can be observed that the best values of the Total Cost Function are obtained when 5 DMAs are introduced. Instead, as expected, a better hydraulic performance is achieved when less reduction of the network connectivity is determined (i.e. with 4 DMAs).

The construction cost is quite far from the adopted budget limit. In particular, 50 of 52 (96%) involve an expenditure lesser than half this limit, while the minimum value is quantified in 15,725 €. However, an interesting outcome is highlighted in Fig. 5.15b, which shows that the minimization of both the objectives requires very low investments, while higher cost is associated to the trade-off solutions.

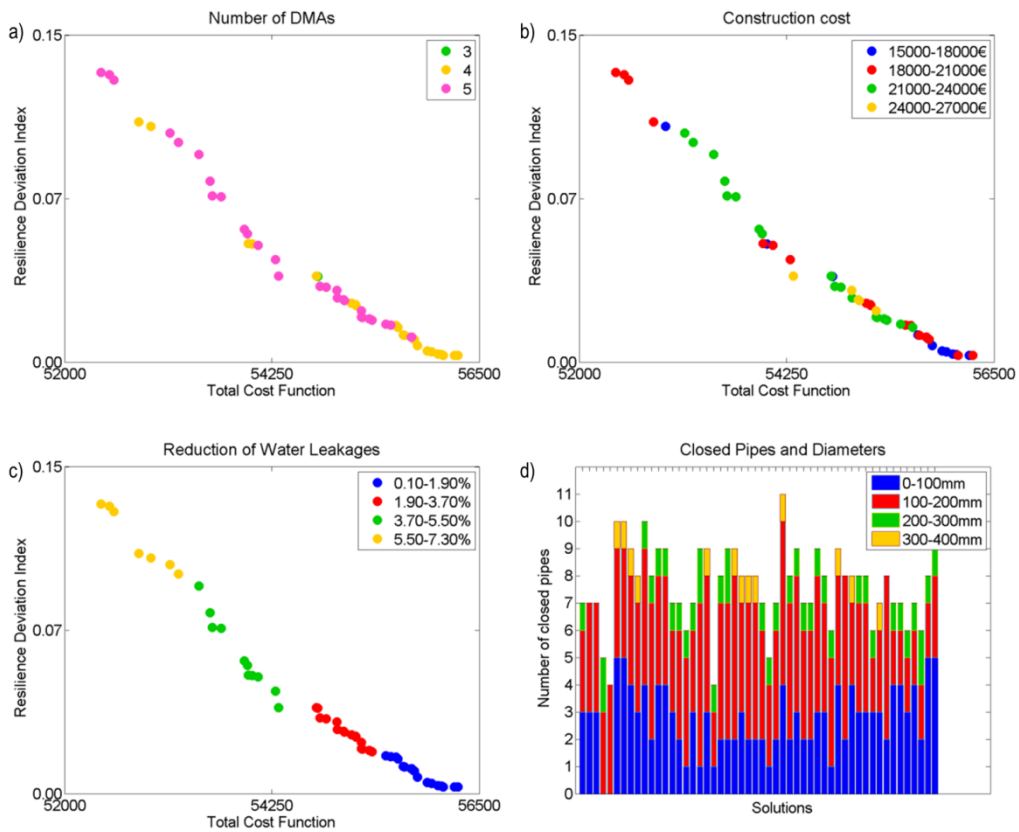


Fig. 5.15 – Sectorization of the WDN of Pianura: technical plots

Similarly to what was pointed out in the analysis of the example network, the reduction of the water leakages shows a clear trend with respect to the values of objective functions (Fig. 5.15c). The effectiveness of the sectorization for the improvement of the network management is emphasized by the remarkable savings that can be achieved in terms of water losses, which range from a minimum of 0.18% up to the 7.16%.

The technical feasibility of the solutions is also attested by the number of pipes that must be closed, which is globally ranged between 4 and 11 and whose most frequent value is 7 (Fig. 5.15d). More than 86% of the intercepted pipes are characterized by a medium size diameter, normally between 100 and 200 mm. However, this range covers over the 50% of the existing pipes, while only the 9.6% has a lesser diameter (see Fig. 5.3). Therefore, the obtained solutions

aim at the delimitation of the DMAs through the closure of the smallest pipes, while the main trunks of the network are generally preserved.

The links that are more frequently targeted for the closure are represented in Fig. 5.16. The colour and the thickness of the lines are assigned according to the criteria discussed in Sec. 4.5.3. It is clearly shown that most of the closures are performed in the central-northern area, and this is consistent with the design criteria and with the adopted objectives. Indeed, the optimization leads to the partitioning of the area where most of the water leakages are located. The resulting reduction of the system connectivity involves lowering of the water pressures and, therefore, the water losses.

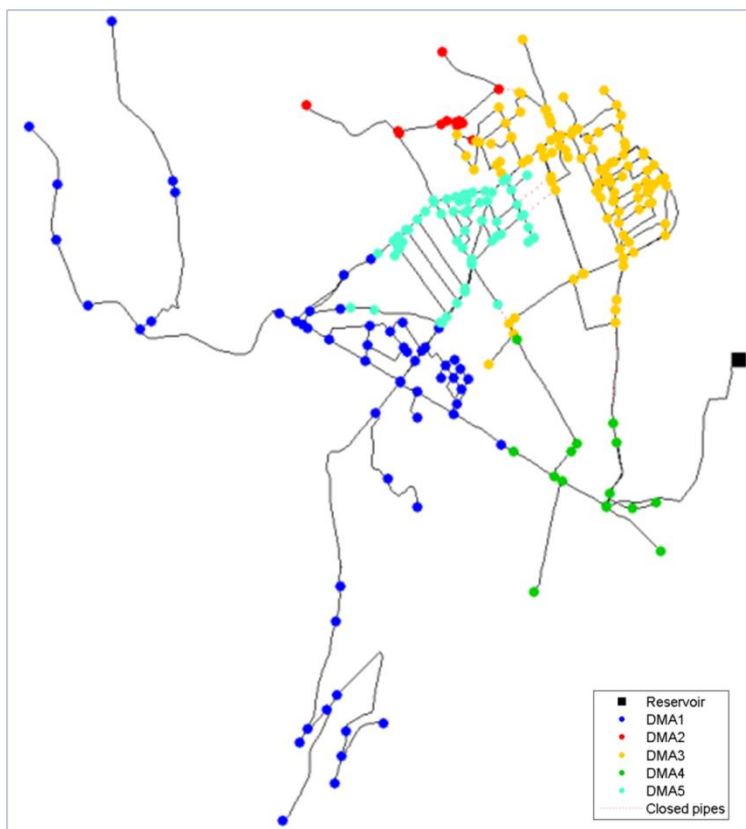


*Fig. 5.16 – Sectorization of the WDN of Pianura: closure frequency of pipes*

Finally, a brief description of three different solutions is provided. First of all, the solution which minimizes the Total Cost Function is considered (Fig. 5.17). It consists of 5 DMAs with different sizes, which are all supplied in cascade. The smallest DMA is located at the upper boundary of the network, that is

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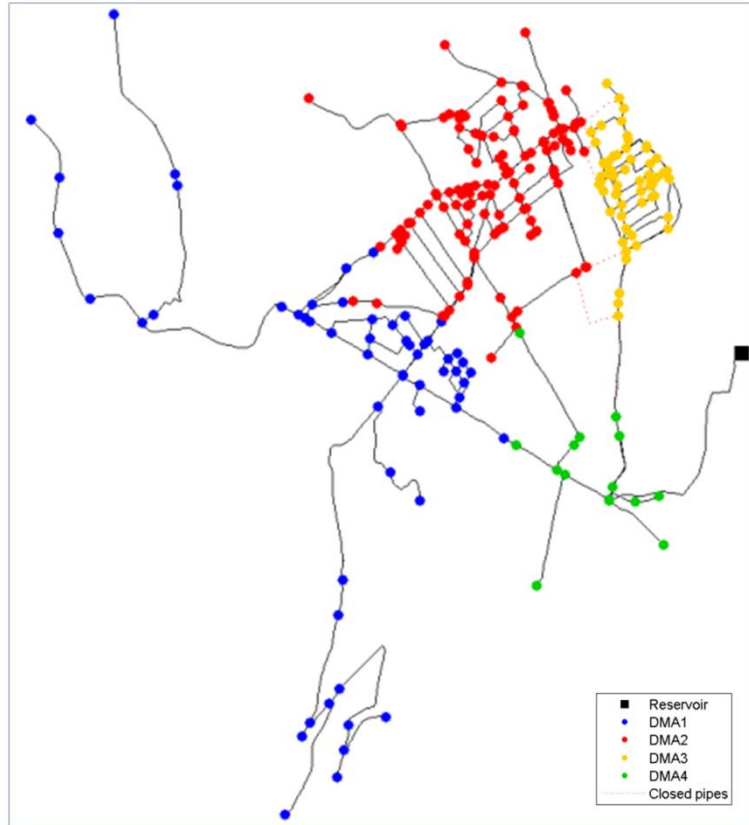
characterized by high ground elevations. The minimum pressure occurs at the northernmost node of DMA 3, and its value is of about 21.12 m. The delimitation of the DMAs entails the closure of 7 pipes, one of which having a diameter of 250, while the others are included in the range between 100 and 200 mm. This involves a construction cost of 19,355 (including the flow-meters) against a 7.16% reduction of the water leakages. This solution also involves the largest loss in the hydraulic reliability of the network, which is estimated in approximately the 13.3%.



*Fig. 5.17 – Solution with the minimum value of Total Cost Function*

The second analyzed solution is the one related to the minimum value of the Resilience Deviation Index (0.0031), and therefore, the one having the best hydraulic performance. The partitioning of the network (Fig. 5.18) consists of 4 DMAs, two of which (i.e. the closest to the tank and the southern one)

practically identical to those designed in the previously described solution. The most densely populated area (at north) is divided into two DMAs (numbers 2 and 3), which are both indirectly supplied through other DMAs.

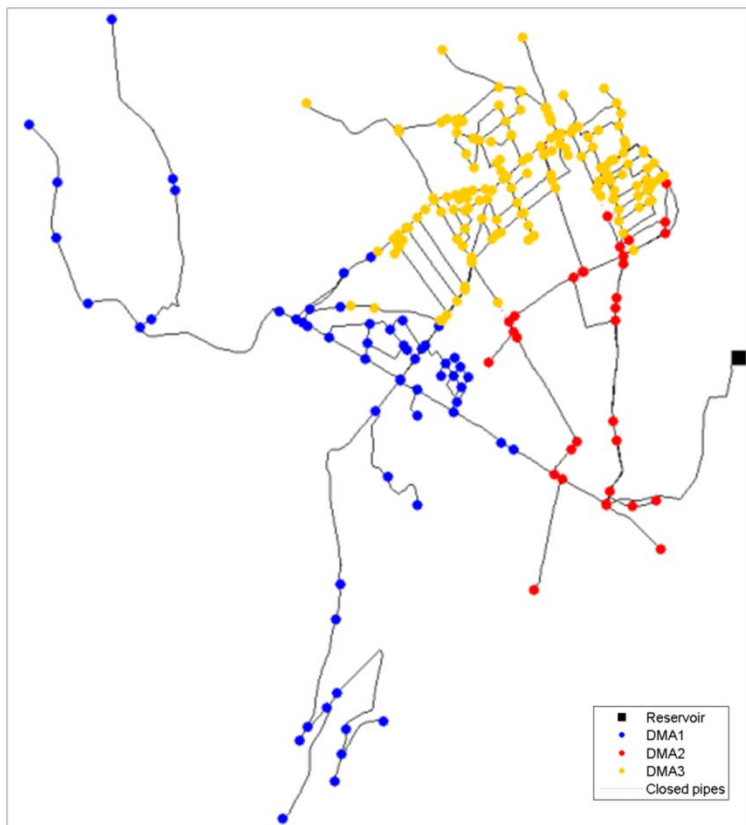


*Fig. 5.18 – Solution with the minimum Resilience Deviation Index*

The water pressure always stays above 28 m and its minimum value (28.16 m) is achieved in DMA 2 during the morning peak hour. Even though this solution is intended to preserve such high pressures, it still ensures a slight reduction of the water leakages quantified in the 0.18%. However, an expenditure of 18,085 € is required for the delimitation of the DMAs, which brings to the final value of 56,268 for the Total Cost Function. This rather high amount of the investment is due to the number of intercepted links, that is even larger than that of the aforementioned solution (9 against 7), and which also involves the closure of a 300 mm diameter pipe.

One of the trade-off solutions is presented here as well, namely that characterized by the lowest number of DMAs. This solution is the only one in the optimal set which considers 3 DMAs, and its layout is shown in Fig. 5.19. The most interesting aspect related to this scenario is the possibility of achieving benefits through the most conservative intervention on the operation of the network.

The value of the Total Cost Function is 54,748. The sectorization is performed through the closure of just 4 pipes with different diameters (100, 150, 200 and 300 mm), thus entailing the lowest construction cost among all the solutions in the Pareto front (15,725 €). On the other hand, the expected reduction of the water losses compared to that of other solutions is quite low (2.79%), but this makes it possible to limit the reduction of the hydraulic reliability to the 3.93%.



*Fig. 5.19 - Solution with the minimum number of DMAs*



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## **Conclusions**

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This thesis addressed the sectorization of water distribution networks. As reported in the introducing chapters, this technique is widely recognized as one of the most proficient strategies for improving the management of such crucial and complex systems. The significant contribution it can provide to the management and the reduction of the water leakages was particularly emphasized in this work (see Chapter 2).

The subdivision of the network into smaller areas not only improves the control on the system operation, but it also involves a reduction of the water pressures that can be proficiently used for mitigating the background leakages. This innovative aspect was specifically taken into account in the definition of the design criteria that were considered in the present study.

The in-depth literature review presented in Chapter 3 was useful to identify the techniques that are most commonly adopted for the optimization of the water systems. With specific reference to the sectorization, the combination of an evolutionary algorithm and of graph theory applications was found to be the best choice for improving the efficiency of the analysis in both terms of use of computational resources and required time.

A comprehensive approach for the optimal sectorization of the water distribution networks was introduced in Chapter 4. The aim was that of providing an automatic and computer based tool for decision aiding in the design of DMAs. In addition, the optimization problem was formulated in order to overcome some of the limitations identified in previous studies, in particular:

- the partial selection of the design criteria among the possible ones;
- the reduced automation introduced in some of the developed algorithms;
- the applicability to both small and large networks.

The adopted methodology follows a multi-objective approach that takes into account several design criteria, which can be grouped into three main categories:

- *topological features*, including the network layout, the water demands, and the ground elevations;
- *economic issues*, namely the minimization of the construction cost and of the cost due to the water leakages;
- *network hydraulics*, in terms of both observance of the minimum service requirements of customers and preservation of the network reliability.

These criteria were all considered in the development of the optimization algorithm, whereas the number of objective functions was limited to two in order to improve the understanding of the optimization process and the readability of the results. A cost function was defined which is related to the total operational cost of the sectorization project. As previously mentioned, in addition to the cost of the devices that are required to set the boundaries of the DMAs (i.e. shut-off valves and flow-meters), the cost of pressure-dependent water leakages is also taken into account.

Another interesting feature which was included in this formulation consists of the combination of the two cost items through a weighted sum. The value of the weights can be suitably chosen in order to privilege the minimization of the short-term investment (i.e. the construction cost) or the maximization of the savings due to the reduction of the leakages, which basically represent long-term paybacks. In this way, the decision maker can put the desired emphasis on both the shares of the operational cost, and therefore he can control the optimization according to his priorities.

On the other hand, the main drawback related to sectorization is considered as well. This is namely the reduction of the network reliability due to the closure of network pipes, which is properly measured through a performance indicator adopted as a second objective.

The implemented algorithm is able to provide a number of optimal solutions to the addressed problem in a completely automatic way once the network model is available. Therefore, the intervention of the analyst is only required in the assignment of the input parameters, most of which are related to essential features of the network operation (e.g. the minimum service pressure for the satisfaction of the water demand).

Nevertheless, the application of the methodology requires particular attention regarding the calibration of the NSGA-II, that is the evolutionary algorithm adopted in this work. This choice was justified by the very good performances shown by this algorithm in several applications in the same field of research. However, its efficiency in the exploration of the objective space is affected by the parameters adopted in the optimization.

Therefore, in order to investigate this issue and to provide further insight into the application of the developed methodology, a simple test case from literature was analysed. A small network consisting of 25 nodes (3 reservoirs) and 37 edges (i.e. pipes) was subjected to 90 tests with different combinations of the NSGA-II parameters. Each test was repeated several times, for an overall number of 432 runs and 3,429,000 evaluations of the objectives. This made it possible to identify the best values to adopt in the optimization, which confirmed the general tendency of evolutionary algorithms in improving the quality of the solution at increasing the number of evaluations.

The framework of a technical evaluation of the results was outlined as well, which was particularly useful for the post-process evaluation of the solutions. This was successfully applied in the analysis of a real-world case, namely the water distribution network Pianura, a neighbourhood of the city of Naples, in Italy. This large example consisted of 269 nodes (1 water source) and 313 pipes,

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and was considered as a more challenging benchmark for the optimization algorithm.

The best Pareto front resulting from the analysis consisted of 52 different designs of the DMAs. Among them, three particular solutions were described, namely those reaching the minimum value of each objective, and an intermediate one corresponding to a good trade-off between them. The success of the optimization was stressed by the decent distribution of the obtained points along the entire Pareto front. Furthermore, the technical evaluation made it possible to recognize the following features of the drawn solutions:

- low cost compared to the assumed budget;
- ease of implementation ensured by the reduced number of pipes to be intercepted, which are also characterized by the smallest diameters;
- remarkable reduction of the water leakages due to the simple closure of network pipes;
- reduced impact on the hydraulic reliability of the system;
- independence of the number of DMAs from that of water sources available in the network.

Moreover, since equal emphasis was given to the construction cost and to the water leakages, another interesting way of analysing the solutions is the evaluation of the payback period of the required investment. The values relevant to the three solutions described in Chapter 5 are reported in Tab. 6.1. Except for that aimed at the maximum preservation of the hydraulic reliability (i.e. the one with the least variations in the water pressures), the other two solutions show that the return of the short-term expenditure could be achieved in a very reduced time.

However, it is important to emphasize that all the calculations were performed under the assumption that the shut-off valves necessary for the

delimitation of the DMAs were not already present in the network. Nevertheless, in real cases they are commonly installed in order to isolate single pipes or network sections during the required maintenance. Consequently, the construction cost estimated during the analysis can be considered as the upper bounds of the required investment, and therefore even further benefits could be expected in both terms of expenditures and payback periods.

Feature	Minimum Total Cost	Minimum Res. Dev. Ind.	Minimum NDMA
Total Cost Function	52,394	56,268	54,748
Resilience Dev. Index	0.1328	0.0031	0.0393
Number of DMAs	5	4	3
Intercepted pipes	7	9	4
Max. Interc. Diameter (mm)	250	300	300
Construction cost (€)	19,355	18,085	15,725
Leak reduction (%)	7.16	0.18	2.79
Payback period (years)	2.4	90.4	5.0

*Tab. 6.1 – Features and payback periods of some remarkable solutions*

## 6.1. Limitations of the methodology

Despite the very good outcomes obtained in the applications, some limitations can be detected in the proposed methodology, namely the following:

- as highlighted several times in the discussion, the adoption of an evolutionary algorithm can significantly influence the performance of the optimization. A sensitivity analysis of the adopted parameters is always recommended in order to assess the actual convergence of the solutions towards the global optimum. In some cases, this may require remarkable computations and post-processing evaluations;
- the comparability with other approaches might not always yield straightforward results because of the different goals pursued in the

- optimization. This issue can be successfully tackled through the estimation of performance indicators related to the objective functions adopted in this case;
- again, the sensitivity of the obtained results towards other parameters such as the unit costs and the weights in the Total Cost Function should be verified case-by-case;
  - the presence of existing shut-off valves is not taken into account in the evaluation of the construction cost. As recalled before, this could represent a further benefit because it would involve lesser expenditures compared to those estimated in the present analysis. However, an optimization performed with the aim of privileging the use of the valves already installed in the network could lead to completely different results;
  - the introduction of a “fuzzy” constraint concerning the water demands was justified by the flexible requirement expressed by the corresponding design criterion. Nevertheless, it negatively affects the mathematical formulation of the problem as it brings some ambiguity in the definition of the domain of feasible solutions;
  - the possibility of providing cascade supply is a value of the developed methodology, as it makes the design of the optimal sectorization of the network independent from the number of water sources. On the other hand, when multiple sources are available, this feature (but, more in general, the sectorization itself) can produce significant changes in the water flows supplied by each of them. Therefore, in some cases, this situation may lead to the violation of the capacity of the water sources. A corresponding constraint could be introduced for taking into account this issue;
  - a similar consideration can be made about the operation of the pumping stations that may be present in the network. In this case, the performance curves of the pumps should also be considered among the problem constraints. Furthermore, in order to ensure a proper efficiency in terms of



energy consumption, the design of the DMAs could be combined with a pump scheduling optimization. To this aim, the operational statuses of the pumps and the cost of energy should be included in the sets of decision variables and objective functions, respectively.

## **6.2. Future improvements of the research**

In addition to overcoming the drawbacks described in the previous section, the following improvements could be implemented in the future development of the research:

- a better characterization of the variation in the system resilience, which could take into account more explicitly the modification of the network layout induced by the sectorization. The creation of the DMAs determines the loss of loops that are mainly responsible of the high reliability of the water distribution networks. In some cases, especially when the pipes with very low flows are closed, the Resilience Deviation Index could not be able to discriminate between the hydraulic performance of a looped structure and a branched one;
- the pressure management through PRVs and/or PATs. This strategy involves the need to characterize the optimal settings of the devices for achieving the best possible pressure regulation while preserving the hydraulic requirements. However, while the use of the PRVs mainly affects the term of the Total Cost Function related to the water leakages, the PATs also ensure significant earnings due to the energy production that should be included in the same objective;
- the staging of the design of DMAs, which is very interesting from a practical perspective. In real cases the implementation of the sectorization should be carried out with particular attention in order to prevent unexpected effects that can be due to the unavoidable uncertainties of hydraulic analysis. In this

regard, the partitioning of the network is usually performed step-by-step, and larger DMAs are subsequently divided into smaller ones. Instead of achieving this with a trial and error approach, it could be interesting to provide the design of a staged sectorization through suitable adjustments of the proposed approach. Both the objective functions should be structured in order to take into account the benefits and the disadvantages evaluated over a given period (e.g. 20 years). In this case, the set of decision variables should be able to represent the actual sectorization of the network at each of the considered time steps. Moreover, the design should be constrained to produce DMAs which cannot change shape and/or size (i.e. the closed pipes cannot be reopened), but that can just be divided into smaller districts.

### **6.3. Publications related to this thesis**

- De Paola, F., Fontana, N., Galdiero, E., Giugni, M., Sorgenti degli Uberti, G., Vitaletti, M. (2014). Optimal design of district metered areas in water distribution networks. *Procedia Engineering*, 70, 449-457.
- De Paola, F., Fontana, N., Galdiero, E., Giugni, M., Savic, D. A., Sorgenti degli Uberti, G. (2014). Automatic multi-objective sectorization of a water distribution network. *Procedia Engineering*, 89, 1200-1207.
- Galdiero, E., De Paola, F., Fontana, N., Giugni, M., Savic, D. A. (2015). Decision Support System for the optimal design of District Metered Areas. *J. Hydroinform.* (under review).

## Example network test plan

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Nomenclature:

- *popsiz*e      size of the population;
- *ngen*          number of generations;
- $\eta_x$             distribution index for crossover;
- $\eta_m$             distribution index for mutation;
- *nruns*        number of runs;
- *NHV*          Normalized Hyper-Volume

Test Case	<i>popsiz</i> e	<i>ngen</i>	$\eta_x$	$\eta_m$	<i>nruns</i>	Objectives Evaluations	Test Numbers					<i>NHV</i>	Ranks
1	10	30	1	1	5	1,500	1	2	3	4	5	0.2378	20
2	10	30	1	10	5	1,500	6	7	8	9	10	0.3674	14
3	10	30	1	20	5	1,500	11	12	13	14	15	0.1760	21
4	10	30	10	1	5	1,500	16	17	18	19	20	0.1659	22
5	10	30	10	10	5	1,500	21	22	23	24	25	0.1659	22
6	10	30	10	20	5	1,500	26	27	28	29	30	0.1527	24
7	10	30	20	1	5	1,500	31	32	33	34	35	0.1659	22
8	10	30	20	10	5	1,500	36	37	38	39	40	0.1659	22
9	10	30	20	20	5	1,500	41	42	43	44	45	0.1659	22
10	10	50	1	1	5	2,500	46	47	48	49	50	0.3526	16
11	10	50	1	10	5	2,500	51	52	53	54	55	0.0611	26
12	10	50	1	20	5	2,500	56	57	58	59	60	0.1659	22
13	10	50	10	1	5	2,500	61	62	63	64	65	0.1659	22
14	10	50	10	10	5	2,500	66	67	68	69	70	0.1659	22
15	10	50	10	20	5	2,500	71	72	73	74	75	0.1659	22

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Test Case	popsize	ngen	$\eta_x$	$\eta_m$	nruns	Objectives Evaluations	Test Numbers					NHV	Ranks
16	10	50	20	1	5	2,500	76	77	78	79	80	0.2978	19
17	10	50	20	10	5	2,500	81	82	83	84	85	0.3652	15
18	10	50	20	20	5	2,500	86	87	88	89	90	0.3001	18
19	10	100	1	1	5	5,000	91	92	93	94	95	0.5419	9
20	10	100	1	10	5	5,000	96	97	98	99	100	0.0611	26
21	10	100	1	20	5	5,000	101	102	103	104	105	0.4893	13
22	10	100	10	1	5	5,000	106	107	108	109	110	0.3419	17
23	10	100	10	10	5	5,000	111	112	113	114	115	0.3419	17
24	10	100	10	20	5	5,000	116	117	118	119	120	0.2978	19
25	10	100	20	1	5	5,000	121	122	123	124	125	0.2978	19
26	10	100	20	10	5	5,000	126	127	128	129	130	0.2378	20
27	10	100	20	20	5	5,000	131	132	133	134	135	0.2378	20
28	30	30	1	1	5	4,500	136	137	138	139	140	0.7440	4
29	30	30	1	10	5	4,500	141	142	143	144	145	0.6892	5
30	30	30	1	20	5	4,500	146	147	148	149	150	0.2378	20
31	30	30	10	1	5	4,500	151	152	153	154	155	0.2378	20
32	30	30	10	10	5	4,500	156	157	158	159	160	0.4893	13
33	30	30	10	20	5	4,500	161	162	163	164	165	0.1760	21
34	30	30	20	1	5	4,500	166	167	168	169	170	0.2978	19
35	30	30	20	10	5	4,500	171	172	173	174	175	0.2378	20
36	30	30	20	20	5	4,500	176	177	178	179	180	0.1760	21
37	30	50	1	1	5	7,500	181	182	183	184	185	0.3526	16
38	30	50	1	10	5	7,500	186	187	188	189	190	0.1653	23
39	30	50	1	20	5	7,500	191	192	193	194	195	0.5329	10
40	30	50	10	1	5	7,500	196	197	198	199	200	0.2978	19
41	30	50	10	10	5	7,500	201	202	203	204	205	0.7440	4
42	30	50	10	20	5	7,500	206	207	208	209	210	0.3674	14
43	30	50	20	1	5	7,500	211	212	213	214	215	0.2378	20
44	30	50	20	10	5	7,500	216	217	218	219	220	0.2378	20
45	30	50	20	20	5	7,500	221	222	223	224	225	0.1659	22
46	30	100	1	1	5	15,000	226	227	228	229	230	0.5441	8
47	30	100	1	10	5	15,000	231	232	233	234	235	0.7486	3
48	30	100	1	20	5	15,000	236	237	238	239	240	0.5441	8

Test Case	popsize	ngen	$\eta_x$	$\eta_m$	nruns	Objectives Evaluations	Test Numbers					NHV	Ranks
49	30	100	10	1	5	15,000	241	242	243	244	245	0.7440	4
50	30	100	10	10	5	15,000	246	247	248	249	250	0.3674	14
51	30	100	10	20	5	15,000	251	252	253	254	255	0.2378	20
52	30	100	20	1	5	15,000	256	257	258	259	260	0.2378	20
53	30	100	20	10	5	15,000	261	262	263	264	265	0.4893	13
54	30	100	20	20	5	15,000	266	267	268	269	270	0.2978	19
55	50	30	1	1	5	7,500	271	272	273	274	275	0.5719	6
56	50	30	1	10	5	7,500	276	277	278	279	280	0.7440	4
57	50	30	1	20	5	7,500	281	282	283	284	285	0.5419	9
58	50	30	10	1	5	7,500	286	287	288	289	290	0.3526	16
59	50	30	10	10	5	7,500	291	292	293	294	295	0.3526	16
60	50	30	10	20	5	7,500	296	297	298	299	300	0.5441	8
61	50	30	20	1	5	7,500	301	302	303	304	305	0.5283	11
62	50	30	20	10	5	7,500	306	307	308	309	310	0.5441	8
63	50	30	20	20	5	7,500	311	312	313	314	315	0.2978	19
64	50	50	1	1	5	12,500	316	317	318	319	320	0.7440	4
65	50	50	1	10	5	12,500	321	322	323	324	325	0.7440	4
66	50	50	1	20	5	12,500	326	327	328	329	330	0.7440	4
67	50	50	10	1	5	12,500	331	332	333	334	335	0.5674	7
68	50	50	10	10	5	12,500	336	337	338	339	340	0.3526	16
69	50	50	10	20	5	12,500	341	342	343	344	345	0.3674	14
70	50	50	20	1	5	12,500	346	347	348	349	350	0.3526	16
71	50	50	20	10	5	12,500	351	352	353	354	355	0.3526	16
72	50	50	20	20	5	12,500	356	357	358	359	360	0.3419	17
73	50	100	1	1	5	25,000	361	362	363	364	365	0.7440	4
74	50	100	1	10	5	25,000	366	367	368	369	370	0.7440	4
75	50	100	1	20	5	25,000	371	372	373	374	375	0.9641	1
76	50	100	10	1	5	25,000	376	377	378	379	380	0.7440	4
77	50	100	10	10	5	25,000	381	382	383	384	385	0.0627	25
78	50	100	10	20	5	25,000	386	387	388	389	390	0.1659	22
79	50	100	20	1	5	25,000	391	392	393	394	395	0.7486	3
80	50	100	20	10	5	25,000	396	397	398	399	400	0.4893	13
81	50	100	20	20	5	25,000	401	402	403	404	405	0.5203	12

<b>Test Case</b>	<b>popsize</b>	<b>ngen</b>	<b><math>\eta_x</math></b>	<b><math>\eta_m</math></b>	<b>nruns</b>	<b>Objectives Evaluations</b>	<b>Test Numbers</b>				<b>NHV</b>	<b>Ranks</b>	
82	100	1000	1	1	3	300,000	406	407	408	-	-	0.7486	3
83	100	1000	1	10	3	300,000	409	410	411	-	-	0.7486	3
84	100	1000	1	20	3	300,000	412	413	414	-	-	0.7486	3
85	100	1000	10	1	3	300,000	415	416	417	-	-	0.7799	2
86	100	1000	10	10	3	300,000	418	419	420	-	-	0.7440	4
87	100	1000	10	20	3	300,000	421	422	423	-	-	0.7440	4
88	100	1000	20	1	3	300,000	424	425	426	-	-	0.7486	3
89	100	1000	20	10	3	300,000	427	428	429	-	-	0.7440	4
90	100	1000	20	20	3	300,000	430	431	432	-	-	0.7440	4