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Real-time optimal control of water distribution systems
Models and techniques, including intermittent supply conditions

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Abstract

The management of a large and complex water supply system is a great task. At the present time, skilled stuff with their experience and judgement is able to control the operation of pumps and valves to ensure the level of service required by costumers. Nevertheless, the growing complexity of water distribution networks and the uncertainty linked to user demand led to the application of an optimal control system (OCS) as viable alternative. An OCS can be developed starting from the definition of three models: a hydraulic network model, a demand forecast model and an optimization model.

Nowadays, to improve the operation of their water distribution systems many water utilities have adopted Supervisory Control and Data Acquisition (SCADA) facilities, which define the existing state of the network and transmit these data to a control centre at regular and short time intervals. Such systems enable operators to monitor pressures and flow rates throughout the water distribution network and to operate various control elements (i.e., pumps and valves) from a central location. The OCS can operate as an independent element of the operating environment (off-line optimization) or directly integrated with a real time control (RTC) system (on-line optimization).

With increasing energy prices, the cost of electricity used for pumping represents the single largest part of the total operational cost in water distribution systems. For this reason, much research has focused on optimizing pump operation schedules. The scheduling of pumps is frequently undertaken in near-real time, in order to minimize cost and maximize energy savings, however this requires a computationally efficient algorithm that can rapidly identify an acceptable solution.

In this thesis, a methodology based on Linear Programming (LP) has been developed for determining the optimal pump schedule. The optimization problem was formulated as a single-objective by considering only the cost derived from pump energy consumption. The resulting model does not guarantee the identification of the global optimum solution of the pump scheduling problem, due to the inaccuracies introduced by linearization. However, it can provide a solution of sufficient quality to be applied in practice. The methodology was tested on two benchmark water distribution networks, with different complexity. Besides stand-alone applications, LP was tested to seed two metaheuristic algorithms showing the potential to be adopted to rapidly determine an approximate, though acceptable, solution which may itself then be subject to further optimization. The resulting hybrid optimization models have revealed to converge more rapidly respect to the traditional metaheuristic algorithms.

In order to verify the LP solution feasibility extended period simulations of the water distribution network were performed by a new hydraulic solver, which was built, as the above mentioned LP model, on MATLAB environment. The solver is able to carried out steady-state simulations by considering demand or pressure driven approaches, to model different devices, such as pumps, pressure reducing valves, check valves, float valves installed on local private tanks, pump as turbines. Hence, in addition to the optimal pump scheduling problem, the solver was successfully applied to analyse water losses and energy recovery solutions with regard to two real-world water distribution networks characterised by intermittent supply.

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Chapter 1.

Introduction

1.1 Background

Due to increasing electricity prices, in the last decades, water utilities have shown growing attention to energy recovery and saving by searching for optimal solutions for energy management in integrated water systems. Each solution is linked to water system characteristics and, in particular, to the resources availability and quality, to the network topology, to the area topography and to the waste water treatments. Each component of the integrated water system contributes differently to the energy balance and some procedures are currently available for identifying the best energetic configuration.

With regard to water distribution systems, the pumping energy cost represents the single largest part of the total operational cost. Even a small overall increase in operational efficiency may result in significant cost savings to the water industries.

The problem of finding the optimal operating strategy is far from simple. Both electricity tariffs and consumers demand can vary greatly through a typical operating cycle; at the same time, minimum water levels have to be maintained in the tank to ensure reliability of supply. The water distribution system computer modelling is a complex and time-consuming process, due to the nonlinear hydraulic behaviour of such system. Finally, the number of possible operating strategies becomes huge for systems with more than a few pumps and tanks.

To cope with the operational optimization problem, several optimization techniques have been applied: linear, nonlinear and dynamic programming, heuristics and metaheuristic algorithms. Most of them, either greatly simplify the complex water distribution system or require significant time to solve the problem, limiting their real-time capabilities.

Operational planning has to be often performed at regular intervals, making very long lasting simulation times undesirable. Short simulation times will also make possible to use the optimization model in emergency situations where the operational plan of the system has to be adjusted in a limited period of time. Faster optimization runs will bring engineers closer to the goal of online operational control, where the system is continually monitored and adjusted to ensure that operational optimality is maintained at all times.

In the present thesis, a real-time optimal control system framework is presented. Among the components which constitute this framework, a hydraulic network model, able to switch from demand to pressure driven analysis, and a single-objective optimization model, based on Linear Programming, as stand-alone or hybrid model, have been developed. Both the hydraulic and the optimization model have been employed in a number of researches, regarding e.g. water losses analysis, energy recovery assessment and optimization problem resolution in water distribution system.

1.2 Aims of Research

The research presented in this thesis deals with models and techniques for the real time optimal control of water distribution systems. The research aims to develop two of the main components of an optimal control system (OCS), the hydraulic network model and the optimization model. The former is suitable to simulate the hydraulic behaviour of the network, once the boundary conditions are defined. A Head Driven Analysis (HDA) was preferred due to its capability to describe the influence of pressure on the leakages flow rate and on the network hydraulic behaviour, resulting from e.g. intermittent supply. The optimization model was developed for a real time control application, to this aim the Linear Programming capabilities were investigated both for stand-alone and hybrid configuration. The latter was performed by coupling LP with two different metaheuristic algorithms, Hybrid Discrete Dynamically Dimensioned Search algorithm and Genetic Algorithm, in order to improve their convergence. The acceleration of stochastic optimization techniques is of particular importance, given that the greater runtimes are common with such optimizations. For this reason LP was tested to seed them, by rapidly determining an approximate, though acceptable, solution which may itself then be subject to further optimization. Both the hydraulic and the optimization model were developed in MATLAB programming environment.

1.2.1 Objectives

The following objectives have been formulated:

- development of an hydraulic network solver, able to describe different operational condition, including intermittent supply;

- investigation of optimization algorithms for the reduction of energy cost in the water distribution systems, for real-time application;
- definition of optimal control system framework which include the above mentioned models.

1.3 Thesis structure

This thesis is arranged in six chapters with three appendices. In particular, it adopts the following structure:

The second chapter, provides a background of the optimal control of water distribution systems, with particular attention on models which characterized it: hydraulic network models, demand forecast models and optimization models. The importance of the Real Time Control is also pointed out in term of cost-effective operational solutions.

Chapter three introduces the hydraulic network modelling approaches and the novel hydraulic solver. This is able to simulate, as well as normal operating condition, peculiar water distribution network behaviour linked to the intermittent water supply.

A new methodology for the optimal control of pump scheduling problem was developed. The description is provided in Chapter four. The stand-alone ability of Linear Programming on solving the optimization problem is compared with an hybrid algorithm which include two different stochastic algorithms.

Chapter five demonstrates the suitability of the techniques introduced in the previous chapter, through their application to a number of small-scale single-objective optimization problems from the literature.

The final chapter details the conclusions that can be drawn from this research and the proposed methodologies, and suggests further avenues of research.

The first Appendix presents the results of several analysis performed through the hydraulic solver with the novel head-driven relationship, with regard to water losses analysis and energy recovery.

The second and the third Appendices show the input files of the benchmark networks used to test the proposed optimization algorithm.

Chapter 2.

Optimal Control of Pumps in Water Supply Systems

2.1 Optimal control system

The management of a large and complex water supply system is a great task. At the present time, skilled staff with their experience and judgement is able to control the operation of pumps and valves to ensure the level of service required by costumers (Jamieson et al. 2007). Nevertheless, the growing complexity of water distribution network and the uncertainty linked to user demand led to the application of an *optimal control system* (OCS) as viable alternative (Leon et al. 2000).

Nowadays, to improve the operation of their water supply systems many water utilities have adopted Supervisory Control and Data Acquisition (SCADA) facilities, which define the existing state of the network and transmit these data to a control centre at regular and short time intervals. Such systems enable operators to monitor pressures and flow rates throughout the distribution network and to operate various control elements (i.e., pumps and valves) from a central location. The optimal control system can operate as an independent element of the operating environment (off-line optimization) or directly integrated with a real time control (RTC) system (on-line optimization). Each approach leads to develop different models characteristics, e.g. whether the off-line optimization can be achieved slowly due to absence of direct connection with remote control, the on-line optimization requires quick response in order to regulate devices properly along the network.

In order to implement in practice a real time OCS, some characteristics should be presented such as robustness, speed, reliability, accuracy and confidence (Reynolds and Bunn 2010). In other words OCS should be able to face quickly different scenarios which occur in a real world water supply system; to take into account the uncertainty linked to the errors in the field measurements; to analyse and represent in details the real system in the model; and to provide results in a range of optimality, which can however change due to the unpredictable operator own risk understanding.

Ormsbee and Lansey (1994) have reported that optimal control systems can be developed starting from the definition of three models: a hydraulic network model, a demand forecast model and an optimal control model. In particular, a calibrated network model is useful to determine the water supply system behaviour respect to different operational strategies. A demand forecast model is used to predict water demand, which will be integrated into the hydraulic network model. Finally the optimal control model, or simply the optimization model, creates optimal control strategies by minimizing an objective function (e.g. electricity cost, water losses) subject to a number of constraints. In the following sections a brief overview of each model is presented with regard to the pump optimal control which is defined as the pump schedules that result in the lowest operating cost associated at the fulfilment of operational and hydraulic constraints. Pump scheduling is then the process of choosing which of the available pumps are to be used with reference to a water supply system and for which periods of the day the pumps are to be run (Mackle et al. 1995).

2.1.1 Hydraulic network model

In order to calculate the cost of specific pump schedule and verify the fulfilment of the optimization problem constraints, a hydraulic network model is needed. According to the problem size and the objective function, it is possible to choose among different hydraulic network model: mass balance, regression, simplified network hydraulics and full hydraulic simulation (Ormsbee and Lansey 1994).

The *mass balance model* considers the system inflow equal to daily demand plus the rate of change in the tank volume. It assumes also that the pressure constraints at the demand nodes are guaranteed by selecting proper tank water levels and exist some pump combinations able to generate the desired water level variation.

The *regression model* describes the network through a set of non-linear equations that can be evaluated starting from either calibrated network model simulations with different boundary conditions (e.g. initial tank levels, user's demands) or from analysis of the actual operating conditions. Respect to a mass balance model it is more reliable, although it can lead to erroneous evaluations when changes occur in water demands or pump heads or tank levels respect to those used to build the model.

The *simplified hydraulics model* approximates the network hydraulics reducing the effect of several components in a single equation or using simple linear model. This approach requires extensive network analysis in order to define the model coefficients.

The *full hydraulic simulation model* solves the governing set of non-linear equations which describe the hydraulics of water distribution network. It requires many data to formulate and calibrate the model. Nevertheless, the full hydraulic simulations are capable to tackle system changes (e.g. demand variation, tanks or pipes out of service), thus showing greater reliability respect to other models. In addition, the computational effort, which was considered the major drawback of this model until ten years ago, is partially reduced due to the improved computer power and the application of more efficient approaches able to catching the knowledge of the hydraulic simulation model in much more efficient form (Jamieson et al. 2007).

2.1.2 Demand forecast model

The increase in the availability of comprehensive SCADA databases has allowed the improvement of demand forecast model formulations. Water demand forecasting is becoming a basic tool for the design, operation, and management of water-supply systems. While long-term forecasting is of interest mainly for planning and design, short-term forecasting is useful in operation and management (Alvisi et al. 2007, Herrera et al. 2010). In this case, several predictive models have been presented for forecasting demand from residential level (Alcocer-Yamanaka et al. 2008, Alvisi et al. 2003, Buchberger and Wells 1996, Buchberger and Wu 1995) to urban area (Alvisi et al. 2007, Qi and Chang 2011, Salomons et al. 2007, Zhou et al. 2002), on daily and hourly time scales basis. The applied techniques to generate hourly forecasts are identical to used techniques to generate daily forecasts, in Bakker et al. (2013) a good review is presented. For the near-real time optimal control of pump systems hourly demand forecasting is commonly used, based on an averaged demand profile or an adaptive demand-forecasting process which is able to face adequately the high variability of demand (Alvisi et al. 2007). In literature several statistical approaches have been proposed able to cope with the stochastic behaviour of user demand, including also economic, demographic and weather factors (Ghiassi et al. 2008).

Although a forecast model is an essential part of the optimal control system, the choice of a particular model does not depend directly on the optimisation algorithm but on the data available and on the hydraulic model applied.

2.1.3 Optimization model

The third component of an optimal control system is the optimization model. It is used to select the values of the decision variables that minimize the total operating cost of the system while satisfying any required system constraints (Ormsbee and Lansey 1994).

The *operating cost* for a pumping system takes into account energy consumption charge and demand charge. The former is linked with the amount of electric energy consumed during the billing period, while the latter refers to the cost associated to the surplus of the energy consumption that occurs during peak time interval. Due to its great variability, the demand charge is either not considered or implicitly addressed via the system constraints. Therefore, only the energy consumption charge is usually included into optimization models. To limit the operating cost, the consumed energy has to be reduced, e.g. by decreasing the water volume pumped, decreasing the total system head, increasing the overall pump efficiency. Moreover, considering the electricity tariff structure, in which is often possible to distinguish a peak and an off-peak tariff period, further cost savings can be achieved by forcing the pumps to work during the less expensive hours.

The s*ystem constraints* associated with an optimal control problem comprise hydraulic and operational requirements. The former are defined by the governing physical laws that describe the hydraulics of the water distribution network; the latter are linked to achievement of an optimal level of service, both for ensuring costumers satisfaction and leakages management, limitation of pump wear, reduction of carbon footprint. As a result, operators will attempt to satisfy some or all constraints while simultaneously determining least cost operations.

In order to define an optimization problem, the *decision variables* may be identified. For a water-supply pumping system, the problem can be formulated using either a direct or indirect approach. In particular, the direct (or explicit, or discrete) formulation leads to use the pump operating times as decision variables; while the indirect (or implicit, or continuous) formulation expresses the cost function in term of surrogate control variable such as tank level, pump station discharges.

2.2 The Real Time Control

As mentioned above, the optimal control system can be directly integrated with a real time control (RTC) system. To sake of clarity in this section a general description of RTC is presented and the difficulties to develop a real time optimal control system for water supply systems are highlighted.

The RTC of water supply systems has certainly become an advance methodology to reduce the investments in infrastructures of existing and new water systems, and an effective tool for water managers to cope with variable operating conditions (Bhattacharya et al. 2003).

In general, the scheme of a control process can be described by control loops: the feed-forward control (disturbance measurement) and feedback (process measurement) control loop. In any control loop several components can be distinguished: sensors which monitor the process evolution, the actuators which influence the process, the controllers which adjust actuators to fulfil the desired value (set-point) with a reasonable tolerance, and data transmission systems that convey data between the different devices. The control loop showed in Figure 2.1 is the basic element of any real time control system. In feedback loop control, instructions are actuated depending on the measured deviation of the controlled process from the set-point. Unless there is a deviation, a feedback controller is not actuated. A feed-forward controller anticipates the immediate future values of these deviations using a model of the process. Then, it activates controls ahead of time to avoid the deviations. A feedback / feed-forward controller is a combination of these two types.

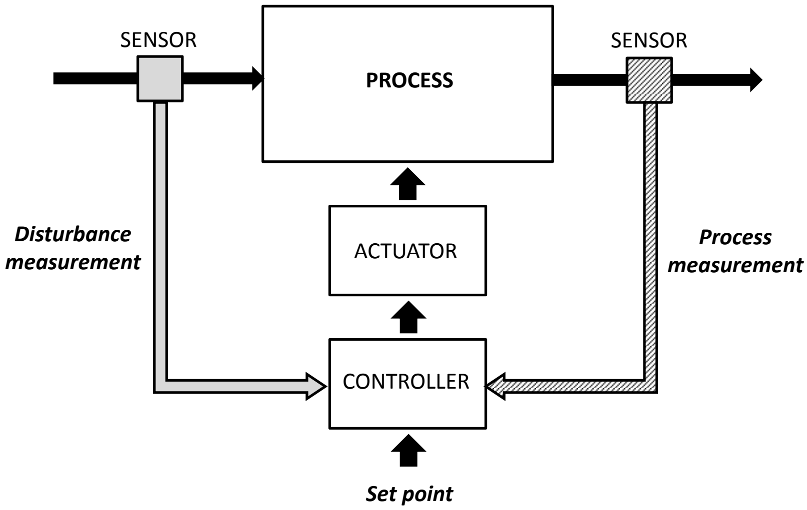

Figure 2.1 The real time control system: the feed-forward control (disturbance measurement) and feedback (process measurement) control loop.

The optimal control of a water supply system can be reasonably designed with feed-forward or feedback/feed-forward controllers. Both configurations allow the development of the optimal control system whose characteristics rely on the accuracy of hydraulic, demand and optimization models and on the reliability, measurement accuracy, suitability for continuous recording and remote transmission, of sensors installed. The physical variables of interest for the management of WDS can be flow rates and pressures over water distribution networks, but also water level in network tanks and chlorine concentration. Starting from the information collected by these devices, the actuators, such as pumps, valves, chemical dosing devices, are set by controllers to accomplish operating objectives.

The proportional integral-derivative (PID) controller is an example of standard controller. It manages the actuator starting from a calibrated function which describes the relation between measured variable and set-point. Nowadays, the digital programmable logic controllers (PLC) are mostly used due to their enclosed functions such as acquisition of measurement data, pre-processing (smoothing, filtering, etc.), checks for status, function, and limits, temporary data storage, calculation of control action, and receive and report data from and to the central station. In the control room, the SCADA system manages all incoming and outgoing data. Data transmission systems may be realised by means of leased or dedicated telephone lines, or by wireless communication systems, such as radio, cellular systems or satellite telecommunication devices.

The difficulties in developing a real time optimal control lies both in improving the accuracy of the models and in the characteristics of the SCADA system (e.g. sampling frequency, memory requirements). The RTC has to operate in very short period of time in order to be effective in the system management. Often a suboptimum control action under the given conditions can be sufficient for RTC, in effect, although several optimization methods are suitable to find the optimal solution, the time constraints for control can be very stringent especially for large network. For this reason many efforts have been made to develop models able to derive quickly optimal control strategy by maintaining reliable and accurate solutions.

2.3 Optimization techniques

In literature several optimization techniques have been applied for the minimization of the operating costs associated with the water supply pumping system: linear (Giacomello et al. 2013, Jowitt and Germanopoulos 1992, Pasha et al. 2009, Price and Ostfeld 2013a), non-linear (Bagirov et al. 2013, Yu et al. 1994) and dynamic programming (Lansey and Awumah 1994, Ulanicki et al. 2007), heuristics (Ormsbee and Reddy 1995) and metaheuristic computations (Lopez-Ibanez et al. 2008, McCormick and Powell 2003, Savic et al. 1997, van Zyl et al. 2004).

Jowitt and Germanopoulos (1992) applied linear programming to a real water distribution network pump scheduling problem. The method was developed on 24 hours basis allowing for unit and maximum demand charges. Linear programming requires that both objective function and constraints have to be a linear combination of the problem variables. The authors introduced a set of assumptions which decoupled pumping station operation from the network hydraulic characteristics. In this way it was possible to eliminate from the problem formulation the non-linear hydraulic relationships. Thus far, the problem can be expressed in linear form except for the maximum demand charge which was considered as well. To evaluate its effect on operating cost, the repeated solution of the linear programming problem was carried out modifying restrictions on the use of the pumping station operating points. The optimal schedule was then selected when the best trade-off between unit and maximum demand charge was achieved. A full hydraulic simulation model was used outside the formulation of the problem to validate the assumptions and the required parameters (e.g. pump flow discharges, power consumptions). Nevertheless, Jowitt and Germanopoulos (1992) argued that linear programming has been shown to be an appropriate technique for real time application, especially when the method effectiveness leads to no need to involve the network extended period simulations. In recent literature, attention has been paid to the applicability of LP to the pumping scheduling optimization problem as well.

Pasha and Lansey (2009) formulated the LP optimization problem linearizing the pumping station relationships by using the relationship between energy, pump flow, user demand and tank water levels. In particular the energy consumed has been approximated as a linear function of the pumping station flow and the initial tank level; the LP model was then tested on a single tank system, although the authors stated that it could be easily extended to more complex systems.

Further investigation into the use of LP algorithm has been reported in Giacomello et al. (2013). Here, a fast, hybrid optimization method was developed, coupling LP with a greedy algorithm which was chosen as the local search method. The former solves a “reduced complexity” hydraulic model, then the latter the “full complexity” hydraulic model: the greedy algorithm performing a search starting from the pumping schedule identified by the LP method. They also demonstrated that the hybrid method, when compared to the pure genetic algorithm optimization method, is capable of solving the real-life pump scheduling problem in a much more computationally efficient manner.

Price and Ostfeld (2013a) presented an iterative convex linearization algorithm for linearization of exponential convex or concave equations with ‘greater than’ and ‘less than’ constraints. Later, they applied this algorithm to solve an optimization model able to investigate the effects of headloss, leakage, pump total head and source cost on minimal cost optimal system operation (Price and Ostfeld 2013b).

An application of non-linear programming can be found in Yu et al. (1994). They proposed a generalized reduced gradient (GRG) algorithm to calculate optimal strategies for daily scheduling, in which maximum demand charges are given a weighting that is proportional to the time elapsed since the beginning of the tariff period. GRG is very powerful for dealing with nonlinear programming with nonlinear constraints. Moreover, the method does not require any network simplification and it can be used for near-real time application in multiple reservoirs and multiple pumping systems, even if the simulator efficiency improvement is needed. The proposed algorithm uses the same accuracy for simulation and optimization model in order to ensure the schedule reliability.

Recently Bagirov et al. (2013) formulated the pump scheduling optimization problem as a mixed integer nonlinear programming problem coupled with a new algorithm based on the combination of a grid search with the Hooke–Jeeves pattern search method. The authors implemented an explicit formulation of pump scheduling, similar to that proposed by Lopez-Ibanez et al. (2008), by introducing the pump start/end run times as continuous variables, and binary integer variables to describe the pump status at the beginning of the scheduling period.

Lansey and Awumah (1994) have determined the optimal pump operations considering the energy and the pump maintenance costs using dynamic programming. The maintenance cost is difficult to quantify, but it can be assumed that it increase as the number of pump switching increases. Hence reducing pump switching can lead to reduce maintenance cost. In this study the pump switching are introduced as a surrogate measure of maintenance cost. Later, several researchers (Baran et al. 2005, Savic et al. 1997) have adopted such approach. Recently, Luo et al. (2012) have considered the pump vibration level as quantification of wear and tear in operation. Lansey and Awumah (1994) have developed two-level approach: a pre-optimization and an optimization step. In the former off-line hydraulic simulations are performed in order to develop the functions describing the network hydraulics and the energy consumption into the dynamic programming algorithm, which constitute the second level. The method showed good results for real-time application, but it is impractical when there are more than three reservoirs. Nevertheless, this limitation can be overcome where large systems consist of a number of small subsystems which are hydraulically independent.

Furthermore in Ulanicki et al. (2007) a two-step water distribution system pumping optimization approach was presented. In particular a generic non-linear programming algorithm was used to minimize operating costs with relaxed constraints. Using solutions from this first stage, the authors were able to use a dynamic programming algorithm to very quickly develop new solutions with different starting conditions or other constraints.

As several researchers before, Ormsbee and Reddy (1995) solved the pump scheduling optimization problem with an explicit formulation, by considering the pump run times instead of the tank level or the pump flow rates. Such approach results in an increase of the number of the decision variables. To cope with this limitation the authors proposed to reduce the variables by rank ordering different pump combinations and developing a single decision variable for each pump station for each control interval. This formulation was solved by coupling nonlinear heuristic with a network simulation model. Heuristic techniques allow to guide the optimization with some specific knowledge. The results showed very efficient computational feature.

As mentioned above, in addition to linear, non-linear and dynamic programming and heuristics, in literature several metaheuristic computation-based models have been proposed to deal with pump scheduling optimization problem solution. Metaheuristic comprises several algorithms such as: Genetic Algorithm, Evolutionary Algorithm, Simulated Annealing, Ant Colony Optimization, Particle Swarm Optimization, Artificial Bee Colony.

Genetic algorithms are nondeterministic algorithms that draws on Darwinian evolution theory. GAs were originally conceived by John Holland in the 1970s, and have since been further developed (Goldberg 2000). The GA methodology is based on the mechanics of natural selection, which combines survival of the fittest with randomized information interchange between the members of a ‘‘population’’ of possible solutions. GAs are best suited to solving combinatorial optimization problems with very large solution spaces which cannot be solved using more conventional optimization methods.

Mackle et al. (1995) have proposed the application of a simple genetic algorithm (GA) to solve the scheduling problem of single pump station with four pump units. The objective was minimising the pumping overall cost over 24 hours gaining from the storage capacity and the availability of off-peak electricity tariff. The method was easy to apply and had produced encouraging preliminary results. Hereafter, Savic et al. (1997) introduced several improvements of this single objective genetic algorithm (SOGA) and also investigated a multi objective genetic algorithm (MOGA) for solving the pump scheduling problem. In contrast to single objective approach, MOGA aims to find a trade-off solution among all objectives. Namely, the minimization of the number of pump switches was also considered, in addition to the main objective of minimising the pump operating cost, by introducing the feasibility of solutions as an additional objective with highest priority. In order to increase the GA performance both SOGA and MOGA approaches were combined with two local search methods based on two different definitions of the neighbourhood of a binary string representing a pump schedule.

Van Zyl et al. (2004) coupled a Genetic Algorithm (GA) to two hill-climbing search algorithms, the Hooke and Jeeves and Fibonacci methods, for improving the local GA search once close to an optimal solution. The hybrid method proved to be superior to the pure GA in finding a good solution quickly, both when applied to a test problem and to a large existing water distribution system. Although these efforts employ evolutionary optimization techniques, operating directly on hydraulic simulation, these cannot cope with near-real time use. The optimization variables were defined in terms of tank level controls. Tank level controls trigger control actions in the distribution system when tank water levels reach certain predetermined values and are widely used in practice, due to their simplicity and proven robustness. The method was applied to a hypothetical water distribution system and a large existing water distribution system.

In Wang et al. (2009), a genetic algorithm-based pump scheduling method was developed aimed to cost reduction and environment protection. Namely, the land subsidence issue linked to groundwater pumping was taken into account. According to the authors such natural resource depletion can be avoided or at least slowed down if groundwater is intermittently pumped. This assumption seems to be in contrast with the reduction of maintenance cost achieved through limitation of pump switches (Lansey and Awumah 1994). Therefore, the authors underlined that a good pumping scheme should be a trade-off between environmental benefit and maintenance cost. As shown in Savic et al. (1997) to improve the convergence speed and the solution quality the randomized initial solutions have to be replaced with particular high-quality initial solutions. Wang et al. (2009) used a greedy algorithm for choosing a good initial solution, according to the water demand and the constraints. Moreover, a local search, named binary local search, was developed according to the properties of the problem in order to enhance the solution quality in each generation.

Genetic algorithm was selected as optimization method also by Luo et al. (2012). They presented a methodology aimed to reduction of energy and maintenance costs. While the traditional approach looks to the reduction on pump switches as a reliable solution for accounting maintenance costs, Luo et al. (2012) introduced the pump machinery vibration as important cause of wear and tear. Based on analysis of the vibration characteristics, an objective model is formulated to describe the relative reliability during operation, which presents a quantitative approach to evaluate alternative operating conditions. This new objective was added to the traditional model to form a new scheduling model. Subsequently, the operating conditions of the pump were improved by making the pump operate at low vibration, the maintenance cost reduced, and the operation reliability enhanced to a certain degree.

Baran et al. (2005) presented an optimal pump-scheduling problem considering four minimisation objectives, it was solved using six multi objective evolutionary algorithms (MOEAs), which were used to solve a test problem with five pumps. These six algorithms were combined with a heuristic algorithm in order to satisfy the problem's constraints, and a mass balance mathematical model. The best suited MOEA was established through a comparison between the algorithms, although the authors recognized some difficulties to perform it due to the variety of parameters affecting each algorithm. Traditional optimisation methods combine all objectives into a single cost function. However, in this study Baran et al. (2005) optimise four objectives simultaneously without aggregation. The selected variables are electric energy cost, pumps' maintenance cost, peak power, and level variation in a reservoir.

McCormick and Powell (2004) outlined a hydraulic network linearization for two stage Simulated Annealing (SA) algorithm. Simulated Annealing is an optimization technique which applies the mutation operator familiar to the Genetic Algorithm to a single solution repeatedly. Initially, a high "temperature" lets the mutation to vary widely the values of the decision variables. As the "temperature" cools, i.e. during the progress of the optimization, the freedom of the mutation to vary the values is constrained – as an analogue with metallurgical annealing in which crystalline solids begin to appear during cooling. Although this technique is able to find a near global optimal solution, it is time consuming and, as a consequence, its application is often limited to off-line optimization problems. Nevertheless, the authors have demonstrated that linear programming can be a viable part of the solution process and that it can accelerate SA optimizations. Model building was based on automatic interaction with a hydraulic simulator and offers potentially wide generality and applicability.

Lopez-Ibanez et al. (2008) have implemented Ant-Colony Optimization (ACO) to minimize electrical cost and, implicitly, pump maintenance costs. ACO operates as an analogue of the essentially random process of ants foraging for food in which individual ants lay pheromone trails as they explore. In the optimization technique, there is a higher probability of an ant following an existing pheromone trail that it encounters of a given threshold strength – resulting in a positive feedback mechanism which allows the "ants" to identify the most direct route to the food source (Dorigo et al. 1996). Rather than represent a pump schedule using binary variables, the authors represented the schedule by defining a series of integers corresponding to the hours each pump will be at on or off state. This schedule was implicitly limited by a "time controlled trigger" integer representing the maximum number of switches between on and off for each individual pump. According to Lansey and Awumah (1994), by limiting the number of pump switches the wear and tear can be reduced as well. Optimal solutions are found by keeping and ranking all solutions according to criteria of descending importance: first, node pressure requirements, then simulation warnings, storage tank volume deficits, and finally low objective function values, i.e. low energy usage. This sorting approach allows the operator to avoid having to set and tweak arbitrary penalty values for each type of violation. According to the authors, this non-penalizing ant-colony approach generates better solutions than a stock genetic algorithm, and does so more quickly.

Recently, Kougias and Theodossiou (2013) applied a music-based metaheuristic method, Harmony Search Algorithm which is inspired to the music creation process in order to find an optimal solution in complex problem.

In summary, many techniques have been applied to solve the pump scheduling optimization problem for water industries. In Table 2.1 the cited references are reported together with hydraulic model, optimization algorithm and decision variables representation used in these control problems. The references listed below cannot be exhaustive due to extensive scientific production on this topic, but it offers a good starting point to further research. Most of mentioned algorithms, either greatly simplify the complex water distribution system or require significant time to solve the problem, e.g. limiting their real-time capabilities. As well as other metaheuristic techniques, although Genetic Algorithms show to be suitable for solving optimization problem, they have not proved they can be used in practice. The scheduling of pumps is frequently undertaken in near-real time, in order to minimize cost and maximize energy savings, however this requires a computationally efficient algorithm that can rapidly identify an acceptable solution. Among optimization techniques hybrid algorithms seem to be the most promising. The hybridization is usually performed by coupling two techniques, the resulting methodology allows to balance the respective drawbacks. In recent years much research has focused on development of models able to cope with different operational problems, taking into account also environmental aspects. Finally, among the methodologies proposed, the choice of the appropriate algorithm for a particular application will be largely dependent on the physical characteristics of the system, but also on the availability of well-calibrated network models and accurate demand forecast models.

Table 2.1 Summary optimization models

Reference	Hydraulic model	Optimization algorithm	Representation	Comments
Jowitt and Germanopoulos (1992)	Simplified Hydraulics	Linear programming (LP)	Explicit	Pump run times are decisions with constant pump output
Yu et al. (1994)	Hydraulic simulation	Non-Linear programming (NLP)	Implicit	Considering the maximum demand charge for the cost evaluation
Lansey and Awumah (1994)	Hydraulic simulation	Dynamic programming (DP)	Explicit	Pump maintenance cost is introduced as constraint
Mackle et al. (1995)	Regression model	Evolutionary algorithm (EA)	Explicit	Binary representation. Minimization of electricity cost and constraints violations on tank levels as penalties
Ormsbee and Reddy (1995)	Hydraulic simulation	Non-Linear Heuristic	Explicit	Custom representation based on rank ordering of pump combinations and percentage of each time interval

Table 2.1 (continued)

Reference	Hydraulic model	Optimization algorithm	Representation	Comments
Savic et al. (1997)	Regression model	Hybrid Genetic algorithm/MOEA	Explicit	Binary representation. Multiple objectives: minimisation of both energy cost and number of pump switches.
McCormick and Powell (2003)	Mass balance	Progressive mixed-integer programming	Explicit	Decision variables are proportion of each pump combination to be used in each time slice.
McCormick and Powell (2004)	EPANET	Simulated annealing (SA)	Explicit	A linearized hydraulic model is iteratively recalibrated using the full model.
van Zyl et al. (2004)	EPANET	Hybrid GA	Implicit	The same algorithm is applied to two networks: a custom network and the Richmond system.
Baran et al. (2005)	Mass balance	MOEA + heuristic constraint algorithm	Explicit	Optimization of four objectives simultaneously without using combined cost function

Table 2.1 (continued)

Reference	Hydraulic model	Optimization algorithm	Representation	Comments
Ulanicki et al. (2007)	Hydraulic simulation	Dynamic programming	Explicit	Two stage optimization
Lopez-Ibanez et al. (2008)	EPANET	Ant colony optimization	Explicit	Maintenance cost is taken into account by “time controlled trigger”
Wang et al. (2009)	Hydraulic simulation	GA	Explicit	Optimization aimed to cost reduction and environment protection
Pasha and Lansey (2009)	-	Linear programming	Implicit	Optimization of pump scheduling and tank level control
Luo et al. (2012)	-	GA	Explicit	Pump machinery vibration as important cause of pump wear and tear

Table 2.1 (continued)

Reference	Hydraulic model	Optimization algorithm	Representation	Comments
Bagirov et al. (2013)	EPANET	Mixed integer nonlinear programming	Explicit	Combination of a grid search with the Hooke–Jeeves pattern search method
Giacomello et al. (2013)	EPANET	Hybrid linear programming	Explicit	Greedy algorithm is used for local search
Price and Ostfeld (2013b)	Hydraulic simulation	Linear programming	Implicit	Analysis of the effects of headloss, leakage, pump total head and source cost
Kougias and Theodossiou (2013)	-	Harmony Search Algorithm	Explicit	The objectives considered: water supply, pumping cost, electric power peak demand and pump maintenance cost.

Chapter 3.

The hydraulic network solver

3.1 Introduction

As above mentioned the hydraulic network model is one of the main components of the optimal control system. Its capability to describe the system hydraulic behaviour is useful both to improve the understanding of system hydraulics and to verify the suitability of operational strategies. Among the models presented in section 2.1.1, the *full hydraulic simulation* model allows to perform satisfactorily reliability analysis, leak detection, water quality analysis and assessment of pumping energy consumption or planning rehabilitation and maintenance practices (Berardi et al. 2010).

At the time of writing, many commercial and open-source software packages are available to perform the hydraulic modelling and analysis of a water distribution system (EPANET, MIKENET and WATERGEMS). Nevertheless, EPANET (Rossman 2000) is certainly considered the industry standard for hydraulic modelling. Unfortunately, its design and programming model have some limitations that make any attempt to extend its hydraulic solver, add new functionalities or improve performance, hard to achieve and time consuming. In the past, some improvements have been introduced to implement new physical elements or process logics by extending the EPANET source code. Among others OOTEN (Cheung et al. 2005), EPANET_EMITTER (Pathirana 2010), CSWNET (Guidolin et al. 2010) can be underlined.

Hydraulic solvers should allow to add and choose different approaches such as pressure driven simulation as well as typical demand driven simulation (see section 3.2); the modelling of existing elements and the support of new hydraulic elements; the possibility to develop and test new functionalities.

In this context, a new hydraulic solver was developed to cope with different water distribution network configurations, e.g. characterized by local tanks, without falling in with EPANET limitations.

The hydraulic solver herein presented performs extended period simulations. It is able to analyse any size of the network, to carried out demand- and pressure-driven simulation; to compute friction headloss using the Hazen-Williams or Darcy-Weisbach formulas; to model pumps, several type of valves (e.g. check valves, pressure reducing valves), pumps as turbines; the storage tanks can have any shape, each node can have its own demand pattern.

In the following, after a brief literature review of the modelling approaches (section 3.2), the description of the mathematical model applied, together with the demand and head loss models are presented (section 3.3). Finally, the different model applications are outlined (section 3.4).

3.2 Modelling approaches

Hydraulic models that simulate the water distribution system behaviour have become an essential tool for design and management. The traditional approach, called Demand Driven Analysis (DDA), assumes that demands along the network are known at time and independent from the pressure in the system, so that the nodal hydraulic head can be determined. Such assumptions lead water utility to estimate demands in order to input these simulation models. As reported in section 2.1.2, several statistical methods are proposed in literature, but also practical guidelines can be applied to this aim.

Most of simulation models based on DDA are suitable to describe network normal conditions, but they are unable to face with abnormal conditions due to pipe breaks, valve failure, pump breakdown or excess of demand. In effect, according to DDA approach the demand is satisfied also when pressure is below to zero that means network can supply water to costumers with low or negative pressure. Clearly, this assumption is unrealistic and represents the major drawback of DDA approach.

In water distribution systems, demands are divided into two types: volume-based and pressure-dependent demands (Wu et al. 2009). While the former are related to e.g. household appliances, industrial process tank, the latter comprise faucets, showers, sprinklers, and leakages. Volume-based demands are unlikely affected by the nodal pressure while pressure-based demands, by definition, are directly dependent upon the available nodal pressure. Some research have been performed in order to determine the proportion between these two types of consumptions. However, as pointed out by Giustolisi and Walski (2012), further investigations are needed.

Since the early 1980s, several studies underlined the necessity to consider the relationship between pressure and discharge at the network node. This kind of analysis is called Head/Pressure Driven Analysis (PDA).

In the following, an overview of the head/pressure driven relationships proposed in literature is showed. For sake of clarity, head dependent demands (head-discharge) and head-leakages relationships are addressed separately.

3.2.1 Head-discharge relationship

Bhave (1981) first considered the dependence of discharge on system pressure. The proposed relation allows full node demand when heads are higher than the minimum value required, conversely no discharges is supposed to be at the node (Figure 3.1a). Later on, Germanopoulos (1985) proposed a head-discharge relationship (Figure 3.1b) to calculate the available outflows at demand nodes described by the following:

$$q_i^{avl} = \begin{cases} q_i^{req} & \text{if} \quad H_i \geq H_i^{des} \\ q_i^{req}\left\{1 - b_i \exp\left[-c_i\left(\dfrac{H_i - H_i^{\min}}{H_i^{des} - H_i^{\min}}\right)\right]\right\} & \text{if} \quad H_i^{\min} < H_i < H_i^{des} \\ 0 & \text{if} \quad H_i \leq H_i^{\min} \end{cases} \qquad 3.1$$

Where q_i^{avl} is the available discharge at node i, q_i^{req} is the required discharge at node i, H_i is the hydraulic head at node i, H_i^{min} is the minimum absolute head at node i, H_i^{des} is the minimum required head, and b_i and c_i are empirical coefficients.

Some issues related to the continuity of this relation were overcome by Gupta and Bhave (1996) as follows:

$$q_i^{avl} = \begin{cases} q_i^{req} & \text{if} \quad H_i \geq H_i^{des} \\ q_i^{req}\left[1 - 10^{-c_i\left(\frac{H_i - H_i^{\min}}{H_i^{des} - H_i^{\min}}\right)}\right] & \text{if} \quad H_i^{\min} < H_i < H_i^{des} \\ 0 & \text{if} \quad H_i \leq H_i^{\min} \end{cases} \qquad 3.2$$

The above mentioned relationships are characterized by an upper bound beyond which the node discharges is supposed to be equal to the required demand. Similar head-demand models were soon after proposed. Wagner et al. (1988) developed a generic pressure-dependent demand through a representation of the orifice relationship (see also Figure 3.1c):

$$q_i^{avl} = \begin{cases} q_i^{req} & \text{if} \quad H_i \geq H_i^{des} \\ q_i^{req}\left(\dfrac{H_i - H_i^{\min}}{H_i^{des} - H_i^{\min}}\right)^{\frac{1}{m}} & \text{if} \quad H_i^{\min} < H_i < H_i^{des} \\ 0 & \text{if} \quad H_i \leq H_i^{\min} \end{cases} \tag{3.3}$$

Where m can vary between 1.5 and 2 as reported in the cited reference.

The method introduced by Reddy and Elango (1989) is wholly different from the others: the pressure-consumption function (Figure 3.1d) does not have an upper bound and the node outflow is the maximum taken by the network, only related to the available nodal pressure:

$$q_i^{avl} = \begin{cases} 0 & if \quad H_i \leq H_{\min} \\ k(H_i - H_{\min})^m & if \quad H_i > H_{\min} \end{cases} \tag{3.4}$$

Where k and m are calibration coefficients. Similar relation was also proposed by Chandapillai (1991).

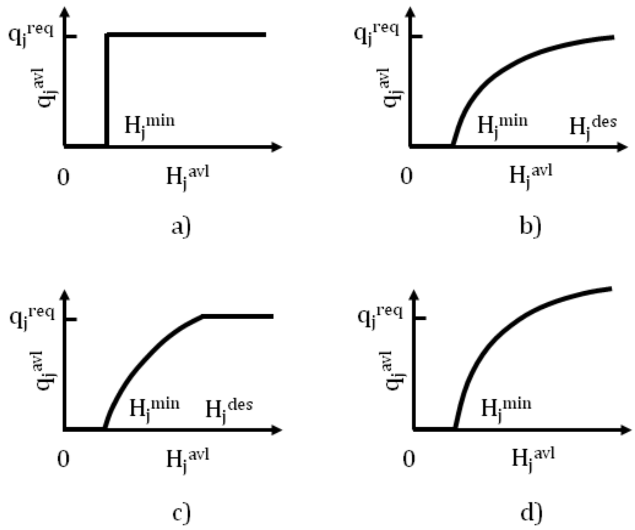

Figure 3.1 Head-driven analysis methods: a) Bhave (1981); b) Germanopoulos (1985); c) Wagner et al. (1988); d) Reddy and Elango (1989)

Fujiwara and Ganesharajah (1993) proposed another relation (eq. 3.5) that has not gained much popularity due to its analytical complexity:

$$q_i^{avl} = \begin{cases} q_i^{req} & \text{if} \quad H_i \geq H_i^{des} \\ q_i^{req} \left[\dfrac{\int_{H_i^{\min}}^{H_i} \left(H_i - H_i^{\min}\right)\left(H_i^{des} - H_i\right) dH}{\int_{H_i^{\min}}^{H_i^{des}} \left(H_i - H_i^{\min}\right)\left(H_i^{des} - H_i\right) dH} \right] & \text{if} \quad H_i^{\min} < H_i < H_i^{des} \\ 0 & \text{if} \quad H_i \leq H_i^{\min} \end{cases} \qquad 3.5$$

Starting from the Wagner's relationship, Wu et al. (2009) introduced (see eq. 3.6) a new parameter, P^{thres}, to take into account the discharges corresponding to values greater than the minimum required pressure:

$$q_i^{avl} = \begin{cases} 0 & \text{if} \quad P_i \leq 0 \\ q_i^{req} \left(\dfrac{P_i}{P_i^{des}} \right)^{0.5} & \text{if} \quad P_i \leq P^{thres} \\ q_i^{req} \left(\dfrac{P^{thres}}{P_i^{des}} \right)^{0.5} & \text{if} \quad P_i \geq P^{thres} \end{cases} \qquad 3.6$$

where P_i is the available pressure at node *i*, P_i^{des} is the desired or minimum required pressure at the node *i*, P^{thres} is the threshold pressure (for values greater than this the nodal discharges is independent of the nodal pressure). This threshold exists for most consumption types, except for leakage discharges.

Criminisi et al. (2009) proposed a relation to describe the node pressure-discharges when local tanks are installed along water distribution networks, between service connections and the user appliances. Starting from Reddy and Elango's formulation, the model was formulated in order to take into account the tank filling/empting process by combining the tank continuity equation and the non-linear float valve closure law. This relationship was slightly modified in De Marchis et al. (2013) to make it suitable for implementation into hydraulic simulation model. Further details are presented forward in section 3.3.3.

Tanyimboh and Templeman (2010) presented the following relationship:

$$q_i^{avl} = \begin{cases} q_i^{req} & \text{if} \quad H_i \geq H_i^{des} \\ q_i^{req} \dfrac{\exp(\alpha_i + \beta_i H_i)}{1 + \exp(\alpha_i + \beta_i H_i)} & \text{if} \quad H_i^{\min} < H_i < H_i^{des} \\ 0 & \text{if} \quad H_i \leq H_i^{\min} \end{cases} \qquad 3.7$$

In which, parameters α_i and β_i can be calculated via either field data calibration or from the following relationships:

$$\alpha_i = \frac{-4.595 H_i^{des} - 6.907 H_i^{\min}}{H_i^{des} - H_i^{\min}} \quad 3.8$$

$$\beta_i = \frac{11.502}{H_i^{des} - H_i^{\min}} \quad 3.9$$

This formula can be easily incorporated into the hydraulic model due to the derivative of this equation has no discontinuity at $H_i = H_i^{des}$ and $H_i = H_i^{min}$, which is a relevant factor in the computational solution of the governing equation which describe the hydraulics of the system.

In recent studies, Shizard et al. (2013) presented the performance of existing pressure-discharges relations by experimental and field measurements of available outflow from different faucets, with fixed opening under various hydraulic pressures. A new pressure discharge relation was also presented and the validity of the relation of Wagner et al. (1988) over other head/pressure driven relationships was confirmed by experimental data. The relationship proposed by Shizard et al. (2013) is as follows:

$$q_i^{avl} = \begin{cases} 0 & if \quad P_f \leq 0 \\ k_f P_f^{0.48} = \dfrac{q_f^{avl(50)}}{\left(P_{\max}^{alw}\right)^{0.48}} P_f^{0.48} & if \quad 0 < P_f \leq P^{thres} \\ k_f \left(P^{thres}\right)^{0.48} = \dfrac{q_f^{avl(50)}}{\left(P_{\max}^{alw}\right)^{0.48}} \left(P^{thres}\right)^{0.48} & if \quad P^{thres} < P_f \end{cases} \quad 3.10$$

q_i^{avl} is the available outflow discharge at faucet; $q_i^{avl(50)}$ is the available outflow discharge at faucet for pressure of 50 m; k_f is the coefficient of pressure at faucet; P_f is the pressure at the faucet; P_{max}^{alw} is the maximum allowed pressure used to calculate the pressure coefficient; and P_{thres} is the threshold pressure above which the outflow discharge is constant. The outflow discharge increases as the pressure increases. However, the authors stated that a pressure threshold can be considered according to the observed pressure discharge curves, of which gradient, for pressures higher than 80 m, was decreased. In reality, head loss in service lines and water meters would be so great that pressure at the faucet would not increase as node pressure increased and hence flow would eventually not increase. This upper bound can be also explained by the fact that user controls demand through the faucets opening to have desired flow rate (Giustolisi and Walski 2012). The relationship in eq. 3.10 was further improved to take into account the impact of volumetric outflows. Respect to the different faucets, the authors have found that the pressure-discharges curves are different for several values of the desired pressure but have a similar shape.

Among the pressure-demand models for controlled outlets proposed in literature, the Wagner's relationship (Wagner et al. 1988) is considered the most feasible to predict WDN pressure-deficient conditions with respect to customer water requests (Giustolisi et al. 2008b, Giustolisi and Walski 2012, Gupta and Bhave 1996, Shirzad et al. 2013). Actually, although Wagner's pressure-demand relationship for customer demands is hydraulically consistent, it is not everywhere differentiable (Tanyimboh et al. 2003). This is a relevant factor in the computational solution of the governing equation which describe the hydraulics of the system. Thus, several methods were developed to assure the differentiability of pressure-demand relationships (Tanyimboh et al. 2003, Tanyimboh and Templeman 2010, Tucciarelli et al. 1999). For the same reason, Giustolisi et al. (2008a, 2008b) introduced an adaptive over-relaxation parameter to pressure-driven analysis within the hydraulic solver and Piller and van Zyl (2010) developed a pressure-driven WDN model using the "content" and "co-content" models.

Finally, many head/pressure driven relationships able to describe controlled and uncontrolled discharges have been proposed in literature. Moreover, the different contribution of volumetric and pressure-dependent demands was taken into account, even if it is not an easy task discerning between them. These relations, as well as having a physical meaning, require a mathematical formulation suitable for implementation within the network hydraulic model. Among the existing formulations, the Wagner's relationship has shown to be suitable to well-describe the node discharges as confirmed by experimental data. However, if WDS are operating under normal conditions and their structural conditions are reasonable known, DDA models can be adopted as hydraulic appraisal tool.

3.2.2 Head-leakage relationship

Several researches showed that pressure is one of the most significant factors influencing leakage in water distribution system (Brunone and Ferrante 2001, Giustolisi et al. 2008b, Lambert 2002, Trow et al. 2004, van Zyl and Clayton 2007). Therefore, it is important to take pressure dependent leakage into account. A leak can be compared to an orifice, for which the well-known Torricelli equation (eq. 3.11) can be adopted to describe the relationship between flow rate and pressure head.

$$q = C_L a \sqrt{2gH} \tag{3.11}$$

where C_L is the discharge coefficient, a is the effective area of the leak, g is the gravity acceleration and H is the hydraulic head.

In steady-state condition, the general equation, that includes the Torricelli equation, can be expressed by eq. 3.12:

$$q_{leak} = aH^b \tag{3.12}$$

$$a = C_L a(2g)^{\frac{1}{2}}, \quad b = \frac{1}{2} \tag{3.13}$$

Some authors showed that the orifice equation can lead to misleading results when the pipe is not made of a rigid material (Ferrante et al. 2011, Greyvenstein and van Zyl 2007). Field studies showed that the sensitivity of the leakage rate to pressure can be significantly larger than 0.5 and typically ranges from 0.5 to 2.79, with a mean value of 1.15 (Trow et al. 2004). The value of this exponent depends on the type of leak, pipe material behaviour, soil hydraulics and water demand (Greyvenstein and van Zyl 2007). Various studies about the pressure dependent leakage modelling can be found in literature (among others (Ferrante et al. 2011, Ferrante et al. 2010, Germanopoulos 1985, Greyvenstein and van Zyl 2007, Massari et al. 2012, May 1994, Tucciarelli et al. 1999, van Zyl and Clayton 2007, Vela et al. 1991). A list of papers dealing with modelling based on leak discharge coefficient and leak area can be also found in Puust et al. (2010) which provides a comprehensive review of leakage management.

Several authors included pipe characteristics into head-leakage relationship. Germanopoulos (1985) proposed a relationship (eq. 3.14) including pipe length, L and a comprehensive coefficient, C related to quality, type, age and other specifications of the network. Considering the average of nodal pressures at the start and end nodes of pipe, the head-leakage relationship is as follows:

$$q_{leak} = C \times L \times \left(P^{av}\right)^{1.18} \tag{3.14}$$

Vela et al. (1991) introduced further parameters representative of the pipe size and condition:

$$q_{leak} = C \times L \times D^d \times e^{a\tau} \left(P^{av}\right)^{1.18} \tag{3.15}$$

where D and τ are pipe diameter and age, respectively; d is 1 for (D < 125 mm) and is equal to –1 for (D > 125 mm); and a is a leakage shape parameter which is difficult to evaluate.

After extensive laboratory tests on different pipe materials (uPVC, asbestos cement and steel), Greyvenstein and van Zyl (2007) presented a basic model for the flow rate through a round hole in an elastic pipe (eq. 3.16).

$$q_{leak} = C_d \frac{\pi d_0^2}{4}\sqrt{2g}\left(H^{\frac{1}{2}} + \frac{2c\rho gD}{3tE} H^{\frac{3}{2}} + \frac{c^2\rho^2 g^2 D^2}{9t^2E^2} H^{\frac{5}{2}} \right) \qquad 3.16$$

where C_d is the discharge coefficient, d_o is the original hole diameter, D is the pipe diameter, t the pipe wall thickness, E the elasticity modulus, ρ the density and c a constant. The authors stated that the processes involved in the expanding leak opening are more complex than the simple power relationship normally used to describe leakage. The equation contains the sum of three terms with leakage exponents of 0.5, 1.5 and 2.5 respectively, which seems to tie in well with field and experimental observations.

According to May (1994), who first proposed the concept of fixed and variable leaks, Cassa et al. (2010) presented the following relationship (eq. 3.17) in order to take into account that leak areas increase linearly with pressure. In this study, the behaviours of different types of leak openings (round holes and longitudinal and circumferential cracks) on pressurized pipes were investigated for different pipe materials (uPVC, steel, cast iron and asbestos cement) using finite element analysis.

$$q_{leak} = C_d \sqrt{2g}\left(A_o P^{0.5} + mP^{1.5} \right) \qquad 3.17$$

with P the pressure head, A_0 the initial leak area at zero pressure, m the pressure–area slope, C_d the discharge coefficient and g acceleration due to gravity. The main difference between the two equations is that May suggested that some leaks have fixed areas (with an exponent of 0.5), while others have variable areas (with an exponent of 1.5). The relationship proposed by Cassa et al. (2010) assumes that all leaks have areas that vary linearly with pressure, and that it is only the extent of the variations that differs.

Recently Ferrante et al. (2011) have investigated the relationship between total head inside the pipe and leak outflow for a single leak in a polyethylene pipe. These tests point out that the viscoelastic nature of the pipe material gives rise to a hysteretic behaviour of the investigated relationship, i.e., the outflow depends not only on the synchronous total head but also on the total head time history and variation rate. Moreover, a comparison between different relationships on basis of experimental data best fitting is provided by Ferrante (2012).

Finally, although the great efforts on finding the proper leakage modelling, the question is still open and further research is needed. The interaction between leak, pressure, pipe material behaviour and soil hydraulics has to be investigated both by experimental and field tests.

3.3 Building a network simulation model

The analysis of water distribution system for steady state flows and pressures involves solving a number of linear and non-linear equations which are governed by the mass conservation law to each node and the energy conservation law to each loop. In general such kind of equations system do not have explicit solutions.

Several solvers compute the hydraulic steady-state variables such as pipe water flow and nodal head by means of the Hardy–Cross, Linear Theory or Newton–Raphson techniques.

The *Hardy Cross method* was developed before the advent of computers. It linearizes the set of non-linear equations for application of an iterative loop by loop relaxation procedure. This method allows to solve small network by hand, however, it is not efficient for large networks.

The *Linear Theory method* solves the set of network equations simultaneously for the flows after linearizing the non-linear terms in the head loss equation for each pipe. Implicit is the assumption of an initial estimate of the flow in each pipe which eliminates the need for initialisation.

The *Newton-Raphson method* has been applied to pipe networks by a number of authors. The method is widely used to solves sets of non-linear equations because it usually converges rapidly to the solution. It requires a set of initial solutions of the unknown variables or a starting point. The efficiency of the method is influenced by selection of a good starting point.

Nevertheless, one of the most promising algorithm due to its computational efficiency is the *Global Gradient Algorithm* (GGA) proposed by Todini and Pilati (1988), which was chosen for the development of the new hydraulic solver, forward presented.

In particular, the GGA was derived by applying the Newton-Raphson technique both in terms of nodal heads and pipe flows to the simultaneous solution of the system of equations expressing mass and energy balance. The problem is analytically reduced to the iterative solution of a system of linear equations, which size equals the number of unknown heads.

The following section introduces the numerical model that integrates the GGA formulated by Todini (2003) and recently improved by Giustolisi et al. (2008b), with a pressure-driven model (De Marchis et al. 2013, Puleo et al. 2013), that permits a more realistic representation of the influence on the network behaviour of the presence of private tanks. The solver is able to carry out demand-driven analysis as well. The model reliability was proven by applying it to solve a sample network proposed in literature (Fujiwara and Khang, 1990). The obtained results showed a good agreement with other models (Savic and Walters 1997). Moreover, some devices (e.g. Pressure Reducing Valves, Pumps, Pumps as Turbines) are modelled with the aim to test several operational strategies, e.g. for water losses reduction, energy recovery and optimization. Water distribution systems are simulated in quasi-steady steady-state conditions, i.e. considering steady-state conditions at each time step with variable boundary conditions and water demands.

3.3.1 Network equations

The problem is in determining all flow rates Q in the pipes as well as the unknown heads H at the nodes on assumption of steady-state conditions. Assuming Todini's formulation (Todini 2003) the looped water distribution network problem in matrix form can be formulated as follows:

$$\begin{bmatrix} \mathbf{A_{pp}} & \mathbf{A_{pn}} \\ \mathbf{A_{np}} & \mathbf{A_{nn}} \end{bmatrix} \begin{bmatrix} \mathbf{Q} \\ \mathbf{H} \end{bmatrix} = \begin{bmatrix} -\mathbf{A_{p0}} \cdot \mathbf{H_0} \\ -\mathbf{q}^* \end{bmatrix} \quad 3.18$$

where $\mathbf{Q}$=[n_p,1] is a column vector of n_p unknown pipes flow rates; $\mathbf{H}$=[n_n,1] is a column vector of n_n unknown nodal heads; $\mathbf{H_0}$=[n_0,1] is a column vector of n_0 known nodal heads. $\mathbf{A_{pn}}=\mathbf{A_{np}}^{\mathbf{T}}$ and $\mathbf{A_{p0}}$ are topological incidence sub-matrices of size [n_p,n_n] and [n_p,n_0], respectively, derived from the general topological matrix $\bar{\mathbf{A}}_{\mathbf{pn}}$ =[$\mathbf{A_{pn}}$ |$\mathbf{A_{p0}}$] of size [n_p,n_n+n_0]. The elements of topological matrix $\bar{\mathbf{A}}_{\mathbf{pn}}$, and similarly of the matrices $\mathbf{A_{p0}}$ and $\mathbf{A_{pn}}$, are defined as follows:

$$\bar{A}_{pn}(i,j) = \begin{cases} -1 & \text{if the flow of pipe } j \text{ leaves node } i \\ 0 & \text{if pipe } j \text{ is not connected to node } i \\ +1 & \text{if the flow of pipe } j \text{ enters node } i \end{cases} \quad 3.19$$

$\mathbf{A_{nn}}$=[n_n,n_n] is a diagonal matrix whose the common element is either zero, if the demand of the node is not "pressure-driven", or is a non-linear function of the pressure. The head-discharges can be effectively defined by different formulations, as detailed above in section 3.2.1 and 3.2.2 respectively for user consumption and leakages modelling. Vector $\mathbf{q^*}$ is a [1,n_n] vector whose element is the actual demand if the node is not pressure driven, while is equal to zero in the case of pressure driven node. Finally, $\mathbf{A_{pp}}$=[n_p,n_p] is diagonal matrix whose elements are defined, for j=1,…,n_p, as:

$$A_{pp}(j,j) = R_j \cdot |Q_j|^{n-1} + m_j \cdot |Q_j| + r_j^{pump} \cdot |Q_j|^{\gamma-1} \quad 3.20$$

where R_j is the head loss coefficient which is a function of a pipe's roughness, diameter and length, and n is the exponent which takes into account the flow regime and the headloss relationship employed (see section 3.3.4). The term m_j is the minor loss coefficient, r_j^{pump} is the pump coefficient and γ is the exponent of pump characteristic curve.

In accordance with the shape of the headloss relationship, the system represented by the equation 3.18 may have more than one solution. If that relationship is monotonically increasing function, it can be proved that the solution of system exists and is unique (Todini and Pilati 1988). In order to solve the system of non-linear equations, provided that matrix $\mathbf{A_{pp}}$ does not become singular (i.e. when the heads at the extreme nodes of a pipe are identical and the flow in the pipe is nullified), the Newton-Raphson technique can be used. The iterative scheme can be achieved by differentiating both sides of equation 3.18 with respect to Q and H:

$$\begin{bmatrix} \mathbf{D_{pp}} & \mathbf{A_{pn}} \\ \mathbf{A_{np}} & \mathbf{D_{nn}} \end{bmatrix} \begin{bmatrix} \mathbf{dQ} \\ \mathbf{dH} \end{bmatrix} = \begin{bmatrix} \mathbf{dE} \\ \mathbf{dq*} \end{bmatrix} \qquad 3.21$$

where $\mathbf{D_{nn}}=[n_n,n_n]$ is diagonal matrix, whose elements are either zero, if the demand is not "pressure driven", or are derivatives of the demand non-linear function with respect to the nodal pressure. $\mathbf{D_{pp}}=[n_p,n_p]$ is diagonal matrix, whose elements are derivatives of the head loss function with respect to pipe flow, then the $\mathbf{D_{pp}}$ matrix elements are defined for $j=1,\ldots,n_p$ as:

$$D_{pp}(j,j) = n \cdot R_j \left|Q_j\right|^{n-1} + 2m\left|Q_j\right| + \gamma \cdot r_j^{pump} \left|Q_j\right|^{\gamma-1} \qquad 3.22$$

Assuming a local linearization between the solution at iteration τ and at iteration τ+1 the equations 3.21 can be written:

$$\begin{aligned} \mathbf{dQ} &= \mathbf{Q}^{\tau} - \mathbf{Q}^{\tau+1} \\ \mathbf{dH} &= \mathbf{H}^{\tau} - \mathbf{H}^{\tau+1} \\ \mathbf{dE} &= \mathbf{A_{pp}}\mathbf{Q}^{\tau} + \mathbf{A_{pn}}\mathbf{H}^{\tau} + \mathbf{A_{p0}}\mathbf{H_0} \\ \mathbf{dq*} &= \mathbf{A_{np}}\mathbf{Q}^{\tau} + \mathbf{A_{nn}}\mathbf{H}^{\tau} + \mathbf{q*} \end{aligned} \qquad 3.23$$

The terms **dE** and **dq*** represent the residuals to be iteratively reduced to zero and $\mathbf{Q}^{\tau}$ and $\mathbf{H}^{\tau}$ are flows and head respectively at the iteration **τ**. Substituting for eqs. 3.23 into eq. 3.21 and analytically solving the system of equations, the iterative formulation of the GGA, considering pumps installed, is as follows:

$$
\begin{cases}
\mathbf{A}^{\tau} = \mathbf{A}_{\mathbf{np}}\left(\mathbf{D}_{\mathbf{pp}}^{\tau}\right)^{-1}\mathbf{A}_{\mathbf{pn}} - \mathbf{D}_{\mathbf{nn}}^{\tau} \\
\mathbf{F}^{\tau} = \left[\mathbf{A}_{\mathbf{np}}\mathbf{Q}^{\tau} - \mathbf{A}_{\mathbf{nn}}^{\tau}\mathbf{H}^{\tau} - \mathbf{q}^{*}\right] - \mathbf{A}_{\mathbf{np}}\left(\mathbf{D}_{\mathbf{pp}}^{\tau}\right)^{-1}\left(\mathbf{A}_{\mathbf{p0}}\mathbf{H}_{\mathbf{0}} + \mathbf{A}_{\mathbf{pp}}^{\tau}\mathbf{Q}^{\tau} - \mathbf{H}^{pump}\right) + \\
\qquad - \mathbf{D}_{\mathbf{nn}}^{\tau}\mathbf{H}^{\tau} \\
\mathbf{H}^{\tau+1} = \left(\mathbf{A}^{\tau}\right)^{-1}\mathbf{F}^{\tau} \\
\mathbf{Q}^{\tau+1} = \mathbf{Q}^{\tau} - \left(\mathbf{D}_{\mathbf{pp}}^{\tau}\right)^{-1}\left(\mathbf{A}_{\mathbf{pp}}^{\tau}\mathbf{Q}^{\tau} + \mathbf{A}_{\mathbf{pn}}\mathbf{H}^{\tau+1} + \mathbf{A}_{\mathbf{p0}}\mathbf{H}_{\mathbf{0}}\right)
\end{cases}
\qquad 3.24
$$

The term $\mathbf{H}_{\mathbf{pump}}$ is the vector related to the pump shutoff head. The iterative formulation described in the eqs. 3.24 can be modified to perform a demand- or pressure-driven analysis, by defining properly $\mathbf{A}_{\mathbf{nn}}$, $\mathbf{D}_{\mathbf{nn}}$ and $\mathbf{q^*}$. In particular, in the first case the matrices $\mathbf{A}_{\mathbf{nn}}$ and $\mathbf{D}_{\mathbf{nn}}$ are not considered and $\mathbf{q^*}$ is set equal to specific value corresponding to the actual demand, conversely in the pressure-driven analysis the vector $\mathbf{q^*}$ is zero, and the matrices are expressed as a function of the pressure.

The problem is thus reduced to the inversion of a symmetrical and sparse matrix. The use of sparse matrix operations and indexing is very efficient in a standard-developing environment such as MATLAB (Mathworks 2011), which has been used in this work.

3.3.2 Network devices modelling

In this section a brief description of Pressure Reducing Valves (PRVs) and Pumps as Turbines (PATs) modelling is presented.

3.3.2.1 Pressure reducing valves

A model for PRVs, enhanced from that proposed by Prescott and Ulanicki (2003),was developed. A PRV provides the desired outlet pressure (set point) through an hydraulic control loop which is able to settle the valve opening and closing (Figure 3.2).

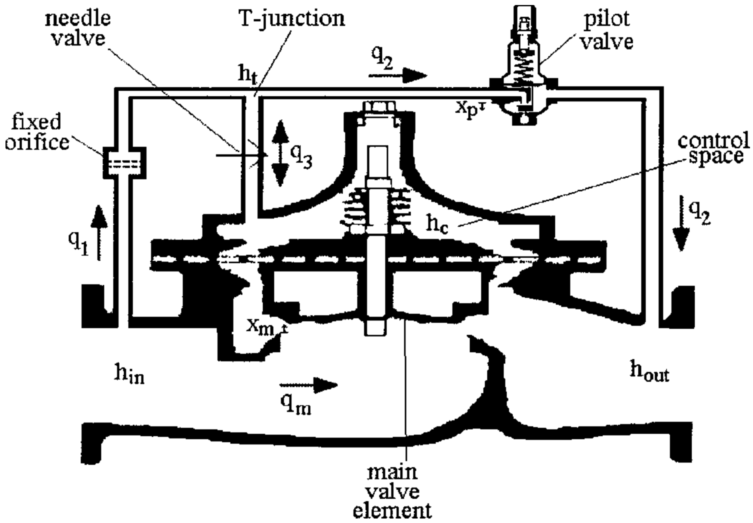

Figure 3.2 Schematic Pressure Reducing Valve (Prescott and Ulanicki 2003)

Therefore for a particular PRV, it is necessary to assess the valve opening as a function of the inlet and outlet pressure values which depend on the network behaviour. The model is based on the following equations:

$$q_3 = \alpha(P_{set} - P_{out}) \tag{3.25}$$

$$\dot{x}_m = \frac{q_3}{A_{cs}(x_m)} \tag{3.26}$$

$$q_m = C_{prv}(x_m)\sqrt{P_{in} - P_{out}} = \frac{1}{m(x_m)}\sqrt{P_{in} - P_{out}} \tag{3.27}$$

where q_3 is flow entering or leaving the valve control space, the chamber above the main valve element; α is the needle valve speed control setting; P_{set} is the PRV set point; $\dot{x}_m$ is the valve opening velocity; A_{cs} is the cross-sectional area of the control space; q_m is the flow passing through the PRV; C_{prv} is the valve capacity; and P_{in}, P_{out} are the PRV inlet and outlet pressure, respectively.

According to Prescott and Ulanicki (2008), it is necessary to specify two relationships: the first (called characteristic curve) between the valve capacity, C_{prv}, and the valve opening, x_m, and the second between the cross-sectional area of the control space, A_{cs}, and the valve opening, x_m. These are provided by the PRV manufactures or can be experimentally measured.

The network and the PRV simulation models are coupled. At each time step for the proper boundary conditions at the junction nodes and network reservoirs, the network model provides the node pressures and the pipe flows; then P_{in}, P_{out} and q_m are determined. For a fixed PRV set-point, q_3, x_m, and

$C_{prv}(x_m)$ are evaluated. The output of PRV model is the PRV minor loss coefficient, *m*, equal to the inverse square of $C_{prv,}$, according to eq. 3.27. Finally, the minor loss coefficient value is fed back to the network model in eq. 3.22 for the next time step.

3.3.2.2 Pumps as Turbines

Prior to describe how Pumps as Turbines (PATs) were implemented into the network hydraulic solver, a brief introduction is provided.

PATs are centrifugal pumps working in reverse mode. Recent literature claims that PATs can represent a viable alternative to traditional turbines. PATs do not require complicated and expensive control systems (Nautiyal et al. 2010). Several applications in off-grid and standalone power plants have been proposed (Arriaga 2010), and recently the use of such devices has been suggested for energy production in water supply systems (Arriaga 2010, Carravetta et al. 2012, Fontana et al. 2012, Ramos et al. 2010).

As pointed out by several authors (Derakhshan and Nourbakhsh 2008a, b, Singh and Nestmann 2010, Williams 1994, 1996, Yang et al. 2013), one of the most important limitations of PAT applicability is related to evaluation of the characteristic curves of the pump in reverse operation. Therefore, establishing a correlation enabling the passage from the 'pump' characteristics to the 'turbine' characteristics is the principal challenge in using a pump as a turbine.

Many researchers have presented theoretical and empirical relations for predicting the PAT characteristics at the Best Efficiency Point (BEP). A good literature review has been provided by Nautiyal et al (2010). Unfortunately, the results predicted by these methods are not reliable for all pumps with different specific speeds and capacities. Moreover, the implementation of the analytical procedure is difficult due to the requirement for very detailed data, some of which is patented and available only to the manufacturers (Singh and Nestmann 2010).

Most recent attempts to predict performance of PATs have been made using Computational Fluid Dynamics (CFD). Carravetta et al. (2012), through the comparison between experimental and CFD analysis, proposed a design method based on a Variable Operating Strategy (VOS) to predict the PAT behaviour and to find the optimal solution which maximizes the produced energy in WDNs. Later, Carravetta et al. (2013) extended the VOS methodology to investigate on best economic efficiency between an hydraulic regulation (HR) or an electrical regulation (ER), founding that HR is more flexible and efficient than ER.

Recently, Puleo et al (2013) analysed the PATs application in a real case through the development of an hydraulic model. They investigate on the potential energy recovery from the use of centrifugal PATs in a water distribution network characterized by the presence of private tanks and intermittent service. The results showed that the energy production can be low and also discontinuous, questioning the efficacy of such energy production, highlighting that further studies are required to investigate the possibility and the efficiency of the PATs to recover energy from WDNs. APPENDIX A, reports details about the results of this study.

The PAT behaviour is very complex and it is difficult to find a simple relationship to cover the behaviour of all pumps in reverse mode. Generally, it has been observed that if the PAT works at other than design flow, a relatively rapid drop in efficiency will be seen. Poor performance, then, is often linked to an inappropriate selection of equipment.

Some experimental studies have been presented in the literature to estimate the characteristic curves of PATs. However, some questions still remain, especially regarding the repeatability of prediction accuracy with respect to pumps of different designs and manufacturers (Singh and Nestmann 2010).

Derakhshan and Nourbakhsh (2008a, b) developed a new method for finding out the BEP of a PAT based on the pump's hydraulic specification. Some correlations were also presented for pumps with different impeller diameters but same specific speeds. The authors illustrated a comparison between different methods for finding out the BEP of a PAT. In addition to this, some relations were presented for determining the complete characteristic curve of a PAT based on its BEP. As suggested by Carravetta et al. (2012), once the prototype characteristic and efficiency curve are available, the results may be extended to obtain the characteristic curves of other similar devices of different runner diameters and rotation speeds by using the Suter parameters (Wylie et al. 1993).

In the new hydraulic solver, modelling of the PATs was undertaken considering their characteristic curves. Once turbine operating conditions (head and flow) are identified, the characteristic curves provided by Derakhshan and Nourbakhsh (2008a, b) can be applied together with informations collected by manufacturers' catalogues.

Namely, they showed head number (ψ) and discharge number (ϕ) curves, from lower to higher specific speeds.

$$\psi = \frac{g\Delta H_{pat}}{n_s^2 D_{imp}^2} \tag{3.28}$$

$$\phi = \frac{Q_{pat}}{n_s D_{imp}^3} \tag{3.29}$$

The discharge number (eq. 3.28) and the head number (eq. 3.29) are dimensionless parameters depending on the rotational speed n_s [rps] and on the impeller diameter D_{imp} [m], g is the acceleration due to gravity [m/s^2], ΔH_{pat} [m]

and Q [m^3/s] are head and flow rate through the device, respectively. Several characteristic curves can be obtained by varying the rotational speed n_s and the impeller diameter D_{imp}.

Knowing the diameter D_{imp}, the actual flow through the PAT, Q_{pat}, and the available water head, ΔH_{pat}, the rotational speed, n_s, can be calculated at each model time step, thus obtaining all of the PAT characteristic parameters. The resulting values can be reported and interpolated, i.e. with second order polynomials or above depending on the desired level of approximation. In eq. 3.30 a quadratic function is exemplified:

$$\Delta H_{pat} = a \cdot Q_{pat}^2 + b \cdot Q_{pat} + c \tag{3.30}$$

In order to take into account the installation of PATs into the eqs 3.18, the generic element of matrix $\mathbf{A_{pp}}$ was transformed:

$$A_{pp}(j,j) = R_j \cdot |Q_j|^{n-1} + 2a_j \cdot |Q_j| + b_j \tag{3.31}$$

Where a_j and b_j are the coefficients of the PAT characteristic curve (eq. 3.30) related to the jth pipe. Therefore, the iterative procedure expressed in eqs. 3.24, considering only the PAT installed, poses as follows:

$$\begin{cases} \mathbf{A^\tau} = \mathbf{A_{np}}\left(\mathbf{D^\tau_{pp}}\right)^{-1}\mathbf{A_{pn}} - \mathbf{D^\tau_{nn}} \\ \mathbf{F^\tau} = \left[\mathbf{A_{np}Q^\tau} - \mathbf{A^\tau_{nn}H^\tau} - \mathbf{q^*}\right] - \mathbf{A_{np}}\left(\mathbf{D^\tau_{pp}}\right)^{-1}\left(\mathbf{A_{p0}H_0} + \mathbf{A^\tau_{pp}Q^\tau} + \mathbf{c}\right) + \\ \quad - \mathbf{D^\tau_{nn}H^\tau} \\ \mathbf{H^{\tau+1}} = \left(\mathbf{A^\tau}\right)^{-1}\mathbf{F^\tau} \\ \mathbf{Q^{\tau+1}} = \mathbf{Q^\tau} - \left(\mathbf{D^\tau_{pp}}\right)^{-1}\left(\mathbf{A^\tau_{pp}Q^\tau} + \mathbf{A_{pn}H^{\tau+1}} + \mathbf{A_{p0}H_0}\right) \end{cases} \tag{3.32}$$

With $\mathbf{c}$ column vector of the PAT c coefficients (eq. 3.30).

3.3.3 A new pressure-driven model

In this section a new pressure driven relationship is presented. The proposed model can be suitable to model local tanks distributed along the water distribution network. Such kind of network configuration is very common in Mediterranean area, in which water scarcity conditions have been experienced in the past.

In order to cope with water shortage, water managers often apply intermittent distribution in order to reduce background leakages and water volumes supplied to the users (Cubillo 2004, 2005). Actually, the influence on user's consumption is often negligible: user demand is mostly dependent on social and climatic factors and users adapt their behaviour to store water when water resources are available and use them when the water distribution service is discontinued (De Marchis et al. 2010). Moreover, this practice leads to network operating conditions that are not accounted for in the typical design.

When a continuous system is managed as an intermittent system, the network pressure is often unable to provide a sufficient level of service; as a result, water distribution is inequitable and not homogenous in space and time (Fontanazza et al. 2007, Fontanazza et al. 2008). The consequent hydraulic condition of the network determines competition among users, that compensate for intermittent water service, by adopting private tanks interposed between the network and the users themselves; in this way water is collected during service periods and then redistributed when public water service is not available (Cobacho et al. 2008, Criminisi et al. 2009, Rizzo and Cilia 2005). The water utility tries to distribute limited water resources as efficiently as possible, by splitting the entire network into different zones defined by number of users and supplying each zone with a ration of the available volume for fixed periods of time (usually less than 24 h). Thus, each zone is subjected to a cyclical filling and emptying process, and users must collect as much water as possible during the service period for covering their needs when supply service is not available. Users try to cope with water service intermittency using private tanks, filled by a proportional float valve, that are often over-designed to take into account possible higher water consumption and leakages. Therefore, node water demand does not depend on actual user consumption, but rather on node water heads.

In this context, Criminisi et al (2009) presented a pressure-driven model able to describe node discharges in networks characterized by the presence of local tanks. In Figure 3.3 a schematic of a typical plumbing connection to a private roof tank is showed.

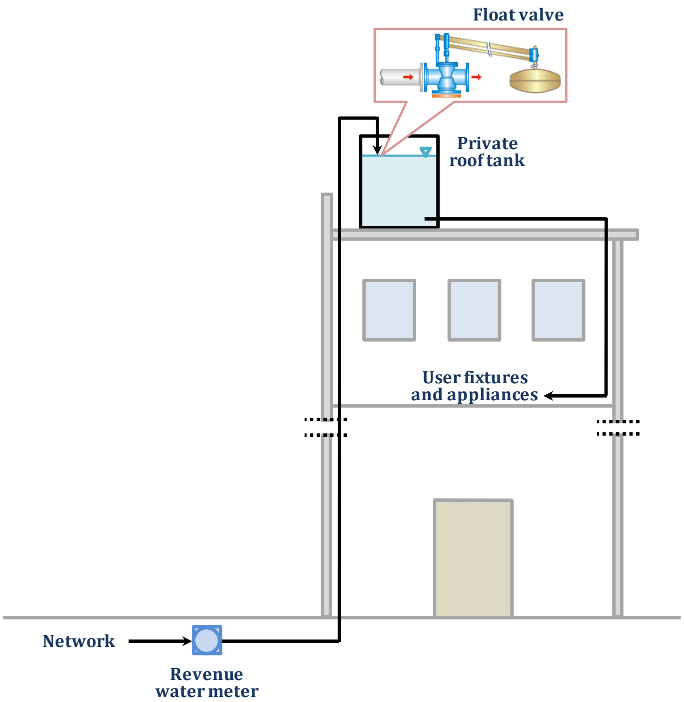

Figure 3.3 A schematic of a typical plumbing connection to a private roof tank (Criminisi et al. 2009)

In particular, the model was based on the use of the float valve emitter law (eq. 3.33) to represent the pressure-consumption relationship at demand node, related to hydraulic head beyond minimum required to have outflow at the tank. In order to take into account the tank filling/empting process, in addition to eq. 3.33, the tank continuity equation was introduced (eq. 3.34):

$$q_{up} = C_v a_v \sqrt{2g\left(P - P_{\text{tank}}\right)} \tag{3.33}$$

$$q_{up} - D = \frac{dV}{dt} = A \cdot \frac{dh}{dt} \tag{3.34}$$

where q_{up} and D are the inflow from the distribution network to the private tank and the user water demand downstream of the tank, respectively; V is the volume of the storage tank having area A and variable water depth h; C_v is the float valve emitter coefficient, a_v is the valve effective discharge area, P is the available pressure at network node, P_{tank} is the available pressure at the tank level; and g is the acceleration due to gravity. Float valve emitter coefficient C_v and the effective discharge area a_v depend on the floater position, and thus on the water level in the tank according to the following empirical laws:

$$C_v = f(h) = \begin{cases} C_v^* & if\ h < h_{\min} \\ C_v^* \left(\dfrac{h_{\max} - h}{h_{\max} - h_{\min}} \right)^n & if\ h > h_{\min} \end{cases} \qquad 3.35$$

$$a_v = f(h) = \begin{cases} a_v^* & if\ h < h_{\min} \\ a_v^* \left(\dfrac{h_{\max} - h}{h_{\max} - h_{\min}} \right)^m & if\ h > h_{\min} \end{cases} \qquad 3.36$$

where h_{min} and h_{max} are the water depths at which the valve is fully open and fully closed [m] (Figure 3.4), respectively, C_v^* and a_v^* are the emitter coefficient [-] and the effective discharge area of the fully open valve [m^2], respectively, and m and n are shape coefficients [-], usually ranging between 0.5 and 2, and must be calibrated.

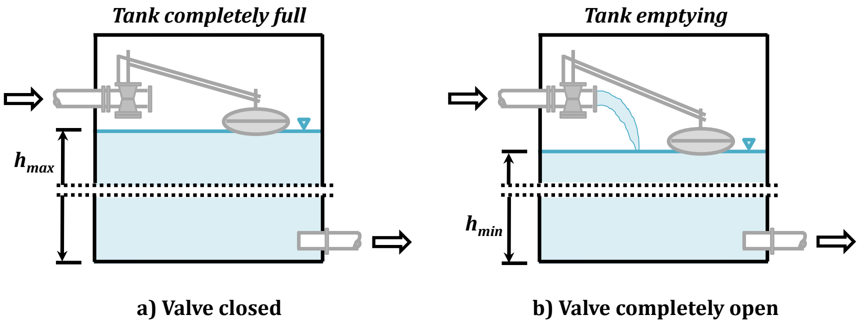

Figure 3.4 Extreme positions of the float valve depending on the tank water level (Criminisi et al 2009)

The authors have considered variation both for emitter coefficient and section area. Actually, the calibration of the coefficients can be considered only for C_v. Moreover, in eqs. 3.35 and 3.36 it should be explicitly specified what happens when tank water level reaches the minimum value selected to activate float valve. Reasonably, the coefficient C_v can be assumed equal to C_v^* at that point. More details about the model and its calibration can be found in the referred referenced literature.

In this report, the formulation presented in Criminisi et al. (2009) was used and slightly modified: an upper bound is introduced beyond which the tank inflow is considered constant and pressure independent, in order to take into account the fact that local head losses in service connections and in water meters greatly increase as flow rates rise. This assumption has to be still validate by experimental tests, although some observations has been already reported in Shirzad et al (2013) for different faucets.

The pressure-driven model provides for each demand node as many tanks as users considered connected to it. Specifically, the discharge entering the kth tank connected to the ith node, q_{act-i}^{k}, at the time *t* (for sake of clarity the t subscript is omitted in the following equations), can be obtained as:

$$q_{act-i}^{k} = \begin{cases} q_{s-i}^{k} & if\ P_i^k > P_{s-i} \\ C_{v-i}^{k} a_{v-i}^{k} \sqrt{2gP_i^k} & if\ P_{\min-i} \leq P_i^k \leq P_{s-i} \quad k = 1,\ldots,N_{\tan k} \\ 0 & if\ P_i^k < P_{\min-i} \end{cases} \qquad 3.37$$

where q_{s-i}^{k} is the maximum discharge entering the k^{th} tank connected to the i^{th} node; C_{v-i}^{k} is the non-dimensional float valve emitter coefficient; a_{v-i}^{k} is the valve effective discharge area; P_i^k is the hydraulic head over the k^{th} tank, otherwise at the network node *i*; *g* is the gravity acceleration; P_{s-i} is the head beyond which node discharge is not influenced by pressure and its values is equal to q_{s-i}^{k}; P_{min-i} is the minimum head required to have outflow at the node; and N_{tank} is the number of tanks connected to the node.

Although more complex methods were considered in the past to relate float valve coefficients to its opening rates, depending on the floater position and thus on the water level of the tank (Criminisi et al., 2009), here constant values were used for C_{v-i}^{k} and a_{v-i}^{k}.

The equation eq. 3.37 must be combined with the tank continuity equation, which can be written for the k^{th} tank connected to i^{th} node as:

$$\begin{cases} q_{act-i}^{k} - dem_i^k = \dfrac{dV_i^k}{dt} = S_i^k \dfrac{dh_i^k}{dt} & if\ h_i^k < h_{\max-i}^{k} \\ q_{act-i}^{k} = 0 & if\ h_i^k \geq h_{\max-i}^{k} \end{cases} \qquad 3.38$$

where dem_i^k is the user water demand; V_i^k is the volume of the k^{th} tank having area S_i^k and variable water level h_i^k; h_{max-i}^{k} is the maximum allowed water level in the tank (before the floating valve closes).

Finally, assuming negligible pressure difference for tanks connected to the same node, the discharge, q_{act-i}, and the water demand of users considered lumped at the i^{th} node, Dem_i, are:

$$q_{act-i} = \sum_{k=1}^{N_{\tan k}} q_{act-i}^{k} \qquad 3.39$$

$$Dem_i = \sum_{k=1}^{N_{\tan k}} dem_i^k \qquad 3.40$$

For the temporal discretization of the continuity equation (eq. 3.38) the first-order Euler method is used. For each node the boundary conditions related to user demand are known and the tank initial volumes are selected according to a random distribution in the range of 10% to 100% of the maximum storage volume, if more accurate informations are not available.

3.3.4 Head-loss relationship

There are a number of head-loss equations that have been developed to determine the frictional losses through a pipe. The three most common equations are the Manning, Hazen-Williams, and Darcy-Weisbach equations. The Manning equation is more typically used for open channel flow and is dependent on the pipe length and diameter, flow, and the roughness coefficient (Manning roughness). Since water distribution networks are constituted by pressurized pipes, this equation is not included into the hydraulic solver.

The Hazen-Williams equation has been used mostly in North America and is distinctive in the use of a *C*-factor. The *C*-factor is used to describe the carrying capacity of a pipe. High *C*-factors represent smooth pipes and low *C*-factors represent rougher pipes. The following is the Hazen-Williams equation:

$$J = \frac{10.67 Q^{1.85}}{C^{1.85} D^{4.87}} \qquad 3.41$$

Where Q is the flow rate and D the pipe diameter.

The Darcy-Weisbach equation was developed using dimensional analysis. This expression uses many of the same variables as the Hazen-Williams equation, but rather than using a *C*-factor it uses a friction factor, f which is a function of relative roughness, Ke and Reynolds number Re (eq. 3.43), for the rough pipes. The following is the Darcy-Weisbach equation:

$$J = f(Ke, \mathrm{Re}) \frac{V^2}{2gD} \qquad 3.42$$

$$\mathrm{Re} = \frac{\rho V D}{\mu} = \frac{VD}{\nu} \qquad 3.43$$

with V velocity, ρ density, μ and ν are the cinematic and dynamic viscosity, respectively.

Several different methods have been developed for estimating the friction factor, f. Two of the main methods are the Colebrook and White (1937) and Swamee and Jain (1976) equations. The Colebrook-White equation is one of the earliest approximation methods that relate the friction factor to the Reynolds number, Re and relative roughness Ke. The following is the Colebrook-White equation:

$$\frac{1}{\sqrt{f}} = -2\log\left(\frac{Ke}{3.71} + \frac{2.52}{\mathrm{Re}\sqrt{f}}\right) \qquad 3.44$$

The main issue with this equation is that the friction factor is found on both sides of the expression. This requires one to solve the expression iteratively to determine which value of the friction factor satisfies the equation. This resulted in the development of the Moody diagram which is a graphical solution for the friction factor. The Swamee-Jain equation is considered to be much easier to solve than the iterative Colebrook-White equation. The following is the Swamee-Jain expression:

$$f = \frac{1.3254}{\left[\ln\left(\frac{Ke}{3.71} + \frac{5.74}{\mathrm{Re}^{0.9}}\right)\right]^2} \qquad 3.45$$

The relative simplicity and accuracy of the Swamee-Jain equation has influenced water distribution system model developers to use this equation to solve for the friction factor.

Recently Sonnad and Goudar (2007) proposed a mathematically exact formulations of the Colebrook–White formula, whose maximum percent error is claimed to be less than 10^{-10}.

In the proposed hydraulic network solver, the Sonnad and Goudar (2007) formulation was applied (eqs. 3.46 – 3.54):

$$u = \ln\left(\frac{d}{q}\right) + \delta \quad \text{or} \quad \frac{1}{\sqrt{f}} = a\left[\ln\left(\frac{d}{q}\right) + \delta\right] \qquad 3.46$$

Where the coefficient q and d are defined by eq. 3.47 and eq. 3.51, respectively:

$$q = s^{[s/(s+1)]} \qquad 3.47$$

$$s = bd + \ln(d) \qquad 3.48$$

$$b = \frac{Ke}{3.7} \qquad 3.49$$

$$d = \left(\frac{\ln(10)}{5.02}\right)\mathrm{Re} \qquad 3.50$$

Whereas, the coefficients a and δ as follows:

$$a = \frac{2}{\ln(10)} \tag{3.51}$$

$$\delta + \ln\left(1 + \frac{\delta}{g}\right) = z \tag{3.52}$$

$$z = \ln\left(\frac{q}{g}\right) \tag{3.53}$$

$$g = bd + \ln\left(\frac{d}{q}\right) \tag{3.54}$$

3.3.5 Convergence criterion

As pointed out by several authors (Giustolisi et al. 2008b), the pressure-driven formulation for q_{act-i} reduces convergence in the Newton–Raphson based algorithms. In fact, the lack of convergence and the need to select a good starting point in eq. 3.24 are the most commonly encountered problems.

In the new hydraulic solver, to overcome the lack of convergence, the following equations were added to the formulation:

$$\mathbf{H}^{\tau+1} = \lambda^{\tau} \cdot \left(\mathbf{H}^{\tau+1} - \mathbf{H}^{\tau}\right) + \mathbf{H}^{\tau} \tag{3.55}$$

$$\mathbf{Q}^{\tau+1} = \lambda^{\tau} \cdot \left(\mathbf{Q}^{\tau+1} - \mathbf{Q}^{\tau}\right) + \mathbf{Q}^{\tau} \tag{3.56}$$

where λ^{τ}, ranging between [0, 1], is an under-relaxation factor to accelerate the solution convergence for a nonlinear hydraulic problem. The under-relaxation factor depends on the progress of the iterations. As the iterations converge, λ^{τ} approaches unity. λ^{τ} is estimated trough convergence by the mean of squared errors in the mass and energy balance equations while performing the iterative search. When any of these errors increases, the value of λ^{τ} is reduced by a factor (set equal to 0.7 here). When both errors decrease, the value of λ^{τ} is increased by a factor (set here equal to 1.5). The maximum number of iterations is also applied as a further control threshold (set here equal to 100).

To select a starting point, the values of $\mathbf{H}^{\tau=0}$ and $\mathbf{Q}^{\tau=0}$ have been chosen as follows:

$$\mathbf{H}^{t=0} = \mathbf{P_{min}} + \mathbf{c_0} \cdot \left(\mathbf{P_s} - \mathbf{P_{min}}\right) + \mathbf{Z} \tag{3.57}$$

$$\mathbf{Q}^{t=0} = \mathbf{pinv}\left(\mathbf{A_{np}}\right) \cdot \mathbf{q}_{\mathbf{act}}^{\mathbf{t=0}} \tag{3.58}$$

where $\mathbf{Z}$ is the vector of nodal elevations; $\mathbf{P_s}$ and $\mathbf{P_{min}}$ are vectors whose elements are P_{s-i} and P_{min-i}, respectively, and $\mathbf{pinv(A_{np})}$ is the Moore–Penrose pseudo-inverse of $\mathbf{A_{np}}$ (a mathematical description is reported in (Giustolisi et al. 2008a); $\mathbf{c_0}$ is a constant whose value ranges between 0 and 1.

3.4 Applications

The proposed network solver was applied to analyse benchmark and real-world water distribution systems.

In particular, the demand driven approach was applied together with an optimization algorithm aimed to reduction of pump energy cost. This application is extensively described in Chapters 4 and 5. Whereas, the pressure driven approach was applied to analyse water losses (leakages and apparent losses) and to test potential energy recovery solutions in water distribution systems characterized by local tanks.

Regarding to real losses, as mentioned in section 3.2.2, several formulations and experimental studies have been proposed in literature to simulate leakages in water networks. Here, the leakage flow rates were accounted by means of the general relationship expressed by eq. 3.12. The selection of leakage parameter values was carried out by elaborations of field data collected by a night flow analysis (Tabesh et al. 2009) in one of the 17 sub network of the water distribution network of Palermo (Italy).

The hydraulic solver was also implemented to identify zones of the network where apparent losses are high and to predict the results of a water meter replacement plan (De Marchis et al. 2013). Apparent losses are financial losses that lead to a decrease in revenue. They consist of water volumes that are withdrawn from the network and consumed by users but not paid for; in developed countries, this nonrevenue water volume may affect the utility's water and economic balances. Water theft and metering errors cause apparent losses (Lambert 2002). Metering errors are typically the main cause of apparent losses and are the most difficult to quantify (Criminisi et al. 2009, Rizzo and Cilia 2005). They derive from intrinsic errors affecting the water meter and depend on the actual flow rate of water passing through the meter. Meter performance is related to technical features of the meter such as correct selection of the meter type and size, proper installation, meter age, wear and tear, the presence of suspended solids in the water (Arregui et al. 2005), the temporal pattern of end user demand (Arregui et al. 2006, Arregui et al. 2007), the presence of user storage tanks (Cobacho et al. 2008, Criminisi et al. 2009, Rizzo and Cilia 2005) and network pressure (Fontanazza et al. 2010a). For further details on apparent losses, it can refer to the cited references. Several methodologies have been proposed in literature to guide water meter replacement strategies (among others: (Arregui et al. 2003, Arregui et al. 2011, Fontanazza et al. 2012).

The influence of pressure control on apparent losses was investigated. PRVs and water metering error model were considered. The PRV model has been already introduced in section 3.3.2.1, while the water metering error model was transferred from a previous experimental and modelling campaign (Criminisi et al. 2009, Fontanazza et al. 2010b). This study was applied to a District Metered Area (DMA) of the water distribution network of Palermo (Italy).

This case study was subjected to further analysis in term of potential energy recovery obtained through the use of PATs (Puleo et al. 2013). As mentioned in section 3.3.2.2, the hydraulic network solver is able to simulate as well as the presence of private tanks and their filling and emptying process, the presence of PATs. These devices were modelled both in the service connections and in the main network pipes.

For sake of clarity, the complete description of the case studies and the results of these applications are organized in APPENDIX A.

Chapter 4.

Pump scheduling optimization

4.1 Introduction

In recent years, much research has focused on optimizing pump operation schedules (section 2.3). With increasing energy prices, the cost of electricity used for pumping represents the single largest part of the total operational cost in water distribution systems. The scheduling of pumps is frequently undertaken in near-real time, in order to minimize cost and maximize energy savings, however this requires a computationally efficient algorithm that can rapidly identify an acceptable solution. Linear Programming (LP), for example, has been shown to be an appropriate technique for this application (Jowitt and Germanopoulos 1992).

In this thesis, a methodology based on LP has been developed for determining the optimal pump schedule for real time application. The resulting model does not guarantee the identification of the global optimum solution of the pump scheduling problem, due to the inaccuracies introduced by linearization. However, it can rapidly provide a solution of sufficient quality to be applied in practice.

As above mentioned, hybrid algorithms have showed to be the most promising among the optimization techniques. For this reason, the ability of the LP to provide "warm" solution or, in other words, an expected domain along the convex curve in which there is a high probability the solution will be found, has been investigated. The hybridization aims to accelerate convergence maintaining a good level of reliability. Therefore, LP solution has been tested as initial seeding into stochastic search algorithms (e.g. Genetic Algorithm, Hybrid Discrete Dynamically Dimensioned Search algorithm), besides being compared with solutions generated by them. In this way, both reliability and suitability of the LP to be used as a pre-optimization stage, have been verified.

This chapter firstly presents a brief overview of the tested algorithms (section 4.2): Linear Programming, Genetic Algorithm and Hybrid Discrete

Dynamically Dimensioned Search algorithm. Secondly, the pump scheduling optimization problem formulation (section 4.3) is described and lastly, some informations about the case studies (section 4.4) are provided.

4.2 Overview of the algorithms applied in this thesis

4.2.1 Linear Programming

Linear programming is one of the most widely used techniques in water resources system management (Simonovic 2009).

LP deals with the problem of minimizing or maximizing a linear function in the presence of linear equality and/or inequality constraints. In addition to the well-known simplex algorithm, the Khachian's ellipsoid algorithm and the Karmakar's projective interior point algorithm are often used to solve linear programming problems (Bazaraa et al. 2011). The latter has inspired a class of interior point methods such as affine scaling methods, primal-dual path-following procedures, and predictor-corrector techniques. Among many general algorithmic approaches, the most effective has proven to be the primal-dual infeasible-interior-point approach, including a number of variants and enhancements such as Mehrotra's predictor-corrector technique (Zhang 1998). Such approach was implemented under MATLAB environment by Zhang (1998) to take advantage of MATLAB's sparse-matrix functions and external interface facilities. In the following the basic definition and the assumptions of linear programming are provided, further details can be retrieved from Bazaraa et al (2011).

Letting z be the *objective function* to be minimized, c_j the *cost coefficients* and x_j the *decision variables* to be determined, with j=1,…,n, the linear programming problem poses as follows:

$$\min(z) = \min(c_1 x_1 + c_2 x_2 + \ldots + c_n x_n) \qquad 4.1$$

Subject to:

$$
\begin{array}{l}
a_{11}x_1 + \; a_{12}x_2 + \ldots + a_{1n}x_n \geq b_1 \\
a_{21}x_1 + \; a_{22}x_2 + \ldots + a_{2n}x_n \geq b_2 \\
\;\vdots \qquad \vdots \quad + \ldots + \; \vdots \quad\; \vdots \\
a_{m1}x_1 + a_{m2}x_2 + \ldots + a_{mn}x_n \geq b_m
\end{array} \qquad 4.2
$$

$$
x_1, x_2, \ldots x_n \geq 0 \qquad 4.3
$$

The inequalities expressed by eq. 4.2 denote the constraints. The coefficient a_{ij} for i=1,…, m, j= 1,…, n are called *technological coefficients*. These coefficients form the *constraint matrix* **A**.

$$
\mathbf{A} = \begin{bmatrix} a_{11} & a_{12} & \ldots & a_{1n} \\ a_{21} & a_{22} & \ldots & a_{2n} \\ \vdots & \vdots & & \vdots \\ a_{m1} & a_{m2} & \ldots & a_{mn} \end{bmatrix} \qquad 4.4
$$

The column vector whose ith component is b_i, which is referred to as the right-hand-side vector, represents the minimal requirements to be satisfied. The constraints in eq. 4.3 are the non-negativity constraints. A set of values of the variables x_j, with j=1,…,n, satisfying all the constraints is called a feasible point or a feasible solution. The set of all such points constitutes the feasible region or the feasible space. Therefore, such a constrained optimization problem may have:

- no feasible solution: there may be no values of all the x_j that satisfy every constraint;
- a unique optimal feasible solution;
- more than one optimal feasible solution;
- a feasible solution such that the objective function is unbounded: the value of the function can be made as large as desired in a maximization problem, or as small in a minimization problem, by selecting an appropriate feasible solution.

The linear programming problem can be stated as: among all feasible solutions, find one that minimizes (or maximizes) the objective function.

To represent an optimization problem as a linear program, several assumptions has to be satisfied:

- *Proportionality*. Given a variable x_j, its contribution to cost is c_jx_j and its contribution to the ith constraint is $a_{ij}x_j$. This means that if x_j is twice, then also its contribution to cost and to each constraint will be twice.
- *Additivity*. The total cost is the sum of the individual costs, and the total contribution to the ith restriction is the sum of the individual

contributions of the individual activities. In other words, there are no interaction or substitution effects among the activities.

- *Divisibility*. The decision variables can be divided into any fractional levels so that non-integral values for the decision variables are permitted.
- *Deterministic*. The coefficients c_j, a_{ij} and b_i are all known deterministically. Any probabilistic or stochastic elements regarding demands, costs, prices, and so on are all assumed to be approximated by these coefficients through some deterministic equivalent.

In summary, if a linear programming problem is being used to model a given situation, then the aforementioned assumptions are implied to hold, at least over some anticipated operating range for the activities. To formulate a linear programming problem, firstly the decision variables, which can vary and affect the value of objective function, are chosen. Secondly, the objective function has to be expressed in term of decision variables. Lastly, the constraints have to be defined in order to restrict the values of decision variables.

4.2.2 Genetic Algorithm

Genetic Algorithms (GAs) belong to evolutionary programs which are probabilistic optimization algorithm based on similarities with the biological evolutionary process (Simonovic 2009). In this concept, a population of individuals, each representing a search point in the space of feasible solutions, is exposed to a collective learning process which proceeds from generation to generation. The population is arbitrarily initialized and subjected to the process of *selection*, *recombination* and *mutation* such that the new populations created in subsequent generations evolve towards more favourable regions of the search space. This is achieved by the combined use of a fitness evaluation of each individual and a selection process resemble the Darwinian rule known as the "survival of the fittest". GAs can operate on multiple objectives simultaneously. Instead of allowing a population of individuals to converge to a single solution, a multiple objective algorithm maintains multiple trade-off solutions for two or more objectives (Goldberg 1989).

GAs have significant advantages: 1) no need for initial solution; 2) easy application to nonlinear problems and to complex systems; 3) production of acceptable results over longer time horizons; 4) generation of several solutions that are very close to the optimum, then the final choice is left to the user.

Therefore, several differences between GAs and more traditional optimization methods can be underlined:

- GAs search a population of points in parallel, not just a single point.
- GAs do not require derivative information or other auxiliary knowledge. Only the objective function and the corresponding fitness levels influence the directions of search.
- GAs use probabilistic transition rules, not deterministic ones.
- GAs are generally more straightforward to apply, because no restrictions for the definition of the objective function exist.

To sake of clarity a brief definition of the process of selection, recombination (or crossover), mutation and reinsertion are provided in the following.

Selection determines which individuals are chosen for recombination and how many offspring each selected individual produces. The first step is fitness assignment by proportional fitness assignment, or rank-based fitness assignment. The actual selection is performed in the next step. Parents are selected according to their fitness by means of one of the following algorithms: roulette-wheel selection, stochastic universal sampling, local selection, truncation selection or tournament selection.

Recombination-crossover produced new individuals by combining the information contained in the parents (parents are the recombination population). Depending on the representation of the variables of the individuals, the following algorithms can be applied: discrete recombination (which is known from the recombination of real-valued variables, and corresponds to uniform crossover of binary-valued variables); intermediate recombination; line recombination; extended line recombination; single-point/double-point/multi-point crossover; uniform crossover; shuffle crossover; and crossover with reduced surrogate.

After recombination every offspring undergoes *mutation*. Offspring variables are mutated by small perturbations (size of the mutation step), with low probability. The representation of the variables determines the algorithm used. Two operators are of importance: a mutation operator for real-valued variables, and a mutation operator for binary-valued variables.

After new offspring are produced they must be inserted into the population. This is especially important if fewer offspring are produced than the size of the original population, or not all offspring are to be used at each generation, or more offspring are generated than are needed. A *reinsertion* scheme determines which individuals should be inserted into the population and which individuals of the population will be replaced by offspring. The used selection algorithm determines the reinsertion scheme: global reinsertion for an all-population-based selection algorithm (roulette-wheel selection, stochastic universal sampling, and truncation selection) and local reinsertion for local selection.

4.2.3 Hybrid Discrete Dynamically Dimensioned Search algorithm

The Hybrid Discrete Dynamically Dimensioned Search (HD-DDS) is a metaheuristic global optimization algorithm (Tolson et al. 2009). One of the major problems associated with the use of such algorithms is that their performance, both in terms of computational efficiency and their ability to find near globally optimal solutions, can be affected significantly by the settings of a number of parameters that control their searching behaviour (e.g., population size, probability of mutation, probability of crossover in the case of GAs), as well as penalty functions that are commonly used to account for system constraints.

Compared with genetic and ant colony algorithms, HD-DDS shows that its searching capability is good while being significantly more computationally efficient. The algorithm's computational efficiency is due to a number of factors, including the fact that it is not a population-based algorithm and only requires computationally expensive hydraulic simulations to be conducted for a fraction of the solutions evaluated. It operates firstly as a global search by permuting decision variables according to a probability distribution, this search is then coupled to a local search method.

4.3 Pump scheduling optimization problem formulation

The pump scheduling optimization problem herein formulated aims to minimize the energy costs, while keeping within physical and operational constraints. According to eq. 4.1, the energy cost function can be defined as sum of the product between power consumption per unit time, that represents the decision variable, and unit time electricity tariff.

The formulation arises as single-objective optimization problem. It is based on a water balance model, then formulated as a linear model and solved by Linear Programming. To reduce the total number of variables, an implicit formulation for the decision variables is selected. In particular, the model determines the optimal operation flow rate for each pumping station for each hour of the simulation period.

According to these assumptions, the objective function is then defined in terms of pump station discharges Q_t (eq. 4.5), and the cost coefficients c_t consider both the electricity tariff and the relation between energy and pump discharge, as discussed below. The optimization period is divided into intervals of one hour. The maximum and minimum water levels into the tank (eq. 4.6), as well as the limit of the pump station duties (eq. 4.7), are considered. Further constraints ensure that the tank level at the end of the optimization period is not lower than the level at the beginning of the next period (eq. 4.8) and the tank mass balance over each control interval (eq. 4.9) is satisfied.

The resulting optimization problem can be then formulated as:

$$\min \sum_{t=1}^{T} c_t \cdot Q_t \qquad 4.5$$

Subject to:

$$S_{\min} \le S_t \le S_{\max} \qquad 4.6$$

$$Q_{\min} \le Q_t \le Q_{\max} \qquad 4.7$$

$$\sum_{t=0}^{T} Q_t = \sum_{t=0}^{T} q_t \qquad 4.8$$

$$Q_t \cdot \Delta t + \left(S_t - S_{s-1}\right) \cdot A = q_t \cdot \Delta t \qquad 4.9$$

where Q_t are the unknown pump station discharges, c_t the objective function coefficients, q_t the known demand, A is the network tank surface area, S_t, S_{t-1} are the tank water level at time t and t – 1 respectively; Δt is the optimization control interval (often fixed to 1 hour), S_{min}, S_{max} are the lower and upper bound referred to the tank water level while Q_{min}, Q_{max} are those related to the pump station discharges.

Prior to solving the system of linear equations, the objective function coefficients c_t have to be evaluated. As mentioned above, in addition to the electricity tariff, these coefficients take into account the network hydraulics, or more precisely, the effect of the water distribution system hydraulic conditions on pump hydraulic power. The pump energy consumption is dependent upon the pump total head which is connected to the pipe resistance curve upstream and downstream of the pump. If the head lift across pump do not vary by more than few meters, pump operates practically at the same point on its pump curve, in spite of variations in the network pressure regime. Conversely, changes may have major effect on the pump energy consumption.

In order to catch these effects, the energy consumption is combined to pump discharge for specific tank initial level and system demand. A generic WDN is simulated for each demand factor, for each pump combination, for a range of water levels. These points are fit with a single regression line for each tank water level and constant demand factor combination. Among the slopes for all pump combinations, the average slope value can be used.

At this point, the objective function coefficients c_t are evaluated as a product of the energy tariff for the average slope of the line interpolating the energy consumptions with respect to pumped flow rates, resulting from the steady-state simulations. The results of these simulations were also used to identify the possible pump combinations (schedules) able to supply the required flow rates resulting from the LP model.

In fact, once a continuous solution is reached by solving the LP problem, it can be converted into discrete pump combination able to provide a similar rate on a 24-hour basis. If the discharge failed to match exactly that of a combination of pumps, some error will result in the approximate cost and tank water level; but these small differences will not significantly modify the optimal solution cost.

An extended period simulation was subsequently undertaken in order to verify the feasibility of the obtained pump schedule.

4.3.1 Stochastic search algorithm seeding

The optimal pump schedule obtained by running LP is used to include a set of warm solutions in the initial generation of the stochastic search algorithms, with the aim to produce an optimization method which is both fast and reliable.

In particular, the LP optimal solution is firstly tested as initial seed solution for the HD-DDS optimization in place of a randomized initialization.

In contrast to the majority of evolutionary optimization techniques, HD-DDS operates on a single solution rather than a population of solutions. It combines a global search method with a local search method developed specifically for the pump scheduling problem, which is executed at several points in the algorithm. The local search element attempts to improve a pumping schedule through changing the status of a single pump at a given time step and re-evaluating the solution. By doing this sequentially, for each of the pumps and time steps, it can be guaranteed that the cost of the pump schedule cannot be improved by switching off any pump at any time during the day without having to simultaneously turn on another pump – and thus increase the cost.

Secondly, the ability of the LP to improve GA convergence is also investigated. LP-derived solutions are used to generate a set of populations. Since LP provides a continuous solution, several schedules can be obtained by selecting different criteria to convert it into a schedule. To increase the number of populations, the schedules derived from the continuous solution are randomly combined by following an uniformly normal distribution.

The cost associated with the result derived from the LP initial solutions is then compared with that obtained with repeated HD-DDS and GA runs with differing random seeds.

4.3.2 Implementation

All the algorithms perform a single-objective optimization, the output is the cost associated to the optimal pump schedule related to well-defined boundary conditions. The optimization models can operate either on stand-alone or hybrid configuration.

The linear programming optimization, together with the hydraulic solver (Chapter 3), is developed into MATLAB environment. In addition to the optimal pump schedule, the LP provides a set of schedules (populations) which can be input into each of the evolutionary algorithms, as discussed above.

Both the evolutionary algorithms are managed through a user interface, which allows to set the parameters that control their searching behaviour (at least for GA), to visualise the evolution of the searching process, to monitor the compliance of the optimised solutions with the constraints and to export the results. Unlike LP, HD-DDS and GA run with EPANET 2 toolkit (Morley and Tricarico 2008) to perform the hydraulic network simulations.

4.4 Test networks

The methodology will be tested on two water distribution networks characterized by different complexity, in order to evaluate its effectiveness and reliability, which are: Anytown test network and a "three pressure-zones" network. Both are designed solely for benchmarking purposes.

4.4.1 Anytown network

The Anytown test network was firstly presented in Walski et al. (1987) as benchmark for finding design able to meet the future water demand. Nowadays, this network is still applied to verify reliability and optimization methods for water distribution networks. Herein, one of the derived networks, whose layout is shown in Figure 4.1, is selected for the purpose of the present study. It is composed of 19 junctions, 1 tank, 37 pipes and 1 reservoir, representing the only external source, from which four different pumps in parallel supply water to the remainder of the system. The demand varies according to a demand pattern with peak factors ranging from 0.4 – 1.2 (Figure 4.2). The daytime tariff cost is set to be twice that charged during the night, namely the peak electricity tariff period ranges from 7:00 to 24:00 and a off-peak tariff from 0:00 to 7:00. The optimization problem is on 24-hours basis. For sake of clarity an EPANET input file describing the network is given in APPENDIX B.1.

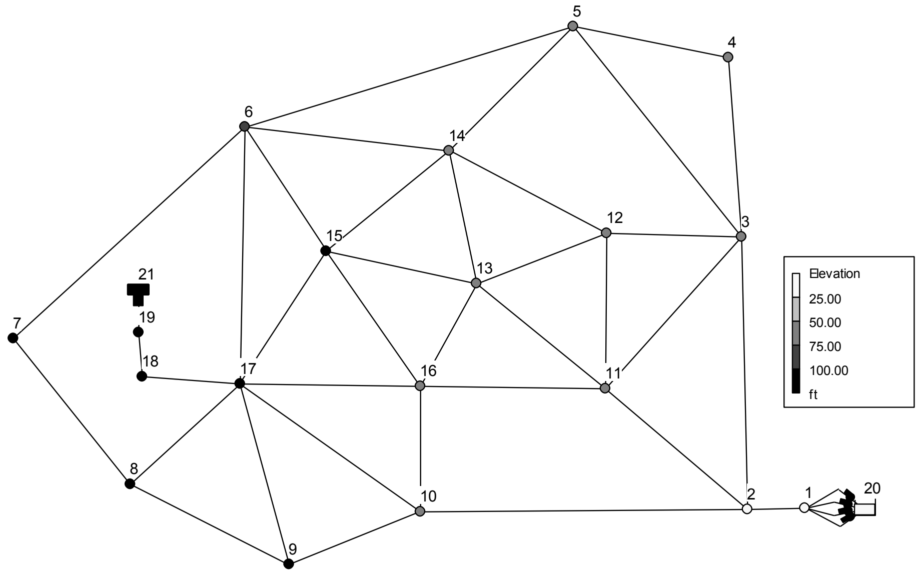

Figure 4.1 Anytown benchmark network

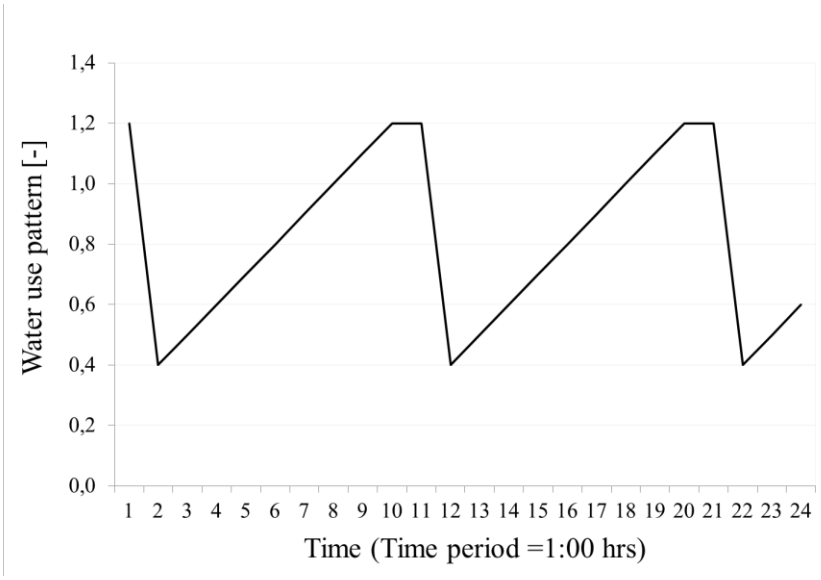

Figure 4.2 Anytown water use pattern

4.4.2 Complex network

Price and Ostfeld (2013a, 2013b) proposed a hypothetical water distribution network to test an iterative linear programming (LP) minimal cost optimal operation supply model. The system seems like water supply in a mountainous area with dense population. It consists of three pressure zones (Figure 4.3), each controlled by a water tank aR, bR and cR. The water sources to the system are from a pumping station aP and from a well aW supplied into pressure “zone a”. From pressure “zone a” two pumping stations bP and cP pump the water to “zone b” and “zone c”. “zone a” consists of four water consumers aD1–4 while “zone b” and “zone c” include one consumer each bD, cD. The consumers demand follows three different water use patterns, correspondingly to each pressure zone. The tariff is grouped into three annual periods: summer (Jul–Aug), winter (Dec–Feb), intermediate (Mar–Jun, Sep–Nov). The tariff is grouped into three daily periods (Sun–Thu, Fri, Sat). According to the session group and the daily period the hours of the day are grouped into three hourly periods: low charge, moderate charge, and peak charge.

The optimization problem is on 168-hours basis. In particular, they solved a weekly problem for each of the months. The complete description of the case study is available in the above mentioned reference. For sake of clarity an EPANET input file describing the network is given in APPENDIX B.2.

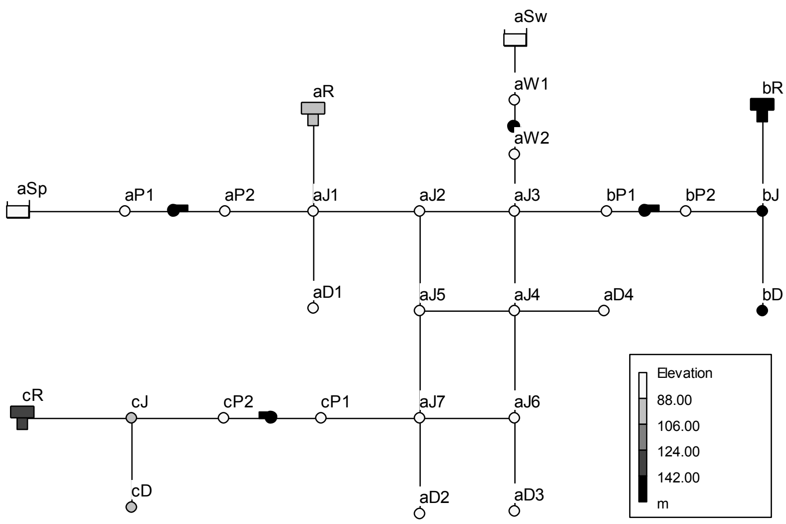

Figure 4.3 Complex network with three pressure zones (Price and Ostfeld 2013b)

Chapter 5.

Optimization solutions analysis

5.1 Linear programming model results

The main issues of the pump scheduling optimization problem (eqs. 4.5 – 4.9) are the constraints' formulation, especially that related to the mass balance equations, and the objective function coefficients evaluation. In particular, the difficulties rely on the evaluation of the network connectivity and the effects of the pipe resistance curves in the pump duty points, respectively. For better understanding, these issues are discussed through the analysis of two water distribution networks characterized by different complexity (see section 4.4).

In particular, with regard to Anytown network, the following aspects were investigated:

- the energy consumption and pump discharge relationship variation, respect to tank initial level and water use demand;
- the error in the linearization of the pump scheduling problem;
- the ability of LP to find an optimal solution even when the tank initial water level changing;
- the impact of the approach for the selection of the pump schedule starting from the LP continuous solution.

Whereas, with regard to the "three pressure zone" network, the suitability of LP to face with complex network, characterized by more than one tank and pump station, was tested.

The Anytown network is characterized by a single tank and a pumping station that comprises four pumps. Then, the formulation of the constraints was easily attained due to the low complexity of the network. According to the methodology above described, the decision variables are the pumping station discharges for each control interval fixed to 1-hour. The optimization problem is on 24-hours basis. Therefore, the evaluation of the objective function coefficients c_j was accomplished by testing all pump combinations, for each water use pattern (Figure 4.2) and for different tank initial level. In the following, for three water use patterns, the pump station discharges and the corresponding energy consumed, resulting from the steady-state simulations with three different tank initial levels (75%, 50% and 25% of the maximum tank water level) are shown (Figures 5.1 – 5.3). The variation between that of the initial level equal to the 50% of the maximum level and the others was about ± 1.3% for the pump flow rate and ± 0.5% for energy consumption. It seems that the variation in the tank initial level has a small influence on the duty point of the pump station compared with that due to variation in demand. Such condition allowed to set arbitrarily the tank initial water level (e.g. equal to 50% of the maximum water level) for all of the steady-state network simulations.

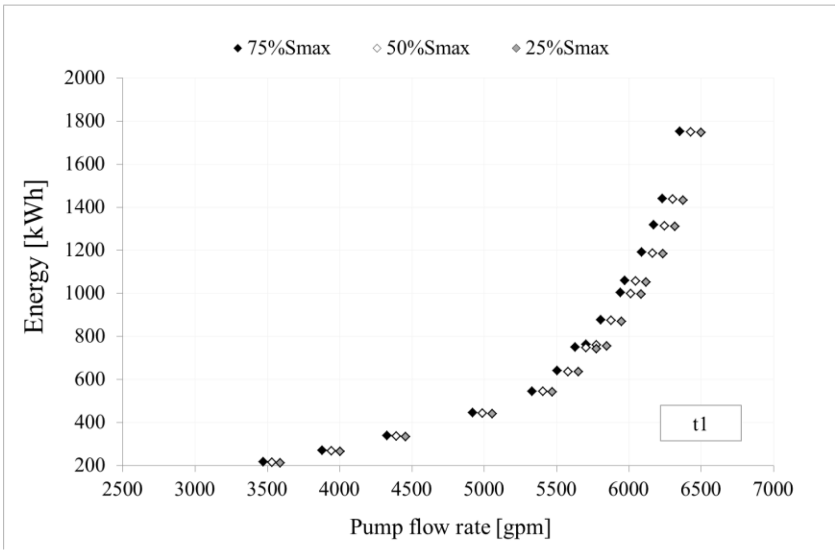

Figure 5.1 Energy consumption related to pumping station flow rate for all the pump combinations (except all pump off) and the water use pattern t1, obtained fixing three different initial tank water levels (75%, 50% and 25% of the maximum tank water level)

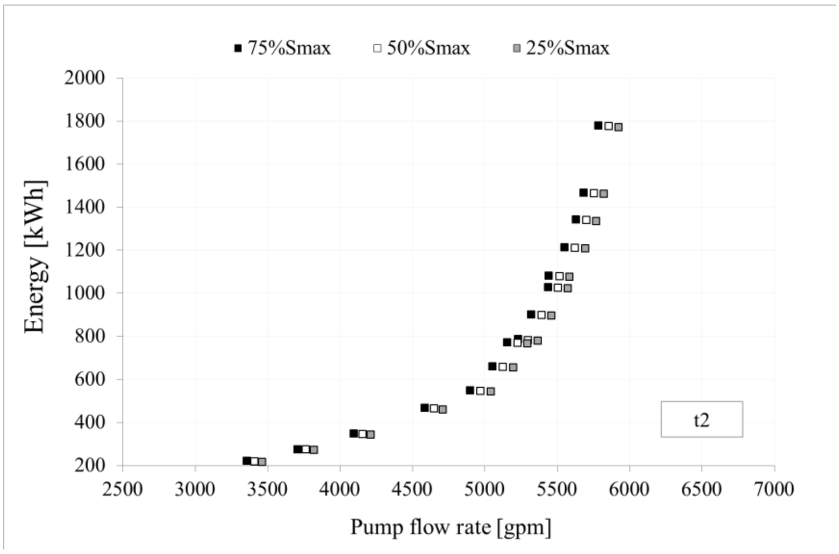

Figure 5.2 Energy consumption related to pumping station flow rate for all the pump combinations (except all pump off) and the water use pattern t2, obtained fixing three different initial tank water levels (75%, 50% and 25% of the maximum tank water level)

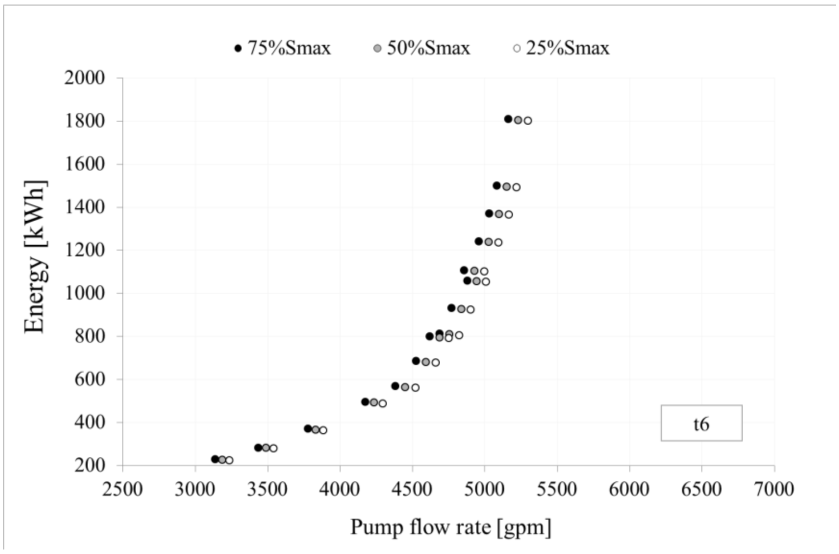

Figure 5.3 Energy consumption related to pumping station flow rate for all the pump combinations (except all pump off) and the water use pattern t6, obtained fixing three different initial tank water levels (75%, 50% and 25% of the maximum tank water level)

Figure 5.4 shows the energy consumption related to pump station discharge obtained from the steady-state network simulations for all pump combinations and demand patterns obtained fixing initial tank level to 50% of its maximum value. The data were interpolated by linear functions in order to be suitable for the linear problem: the slopes of the lines represent the proportional factors between energy and pumped flow. This approximation allows to have an average error of about ±26%, and a maximum error of ±84%. To sake of clarity, among the energy curves presented in Figure 5.4, only a few together with linear and exponential interpolation are listed below (Figures 5.5 – 5.9). The exponential interpolation is reported to point out the awareness of the approximation for using a linear function, instead of a more proper function. Considering Figure 5.5, with the exponential function a pump discharge of about 5000 gpm (gallons per minutes) corresponds to an energy consumption equal to 594 KWh, while with the linear function an overestimate of the energy consumption is obtained (643 KWh). Whereas, when the pump discharge is close to 4000 or 6000 gpm the linear function returns 227.5 or 1058 KWh against 295 and 1196 KWh of the exponential function, in these cases the former underestimates the energy consumption.

Since the slopes do not vary considerably among the different demand patterns, an average value was considered in this analysis. Hence, the objective function coefficients were evaluated. In order to ensure the comparability of the results, the intercept of the energy function, which represents the potential energy linked to available pump total head, was used to update the cost obtained through the solution of the LP problem.

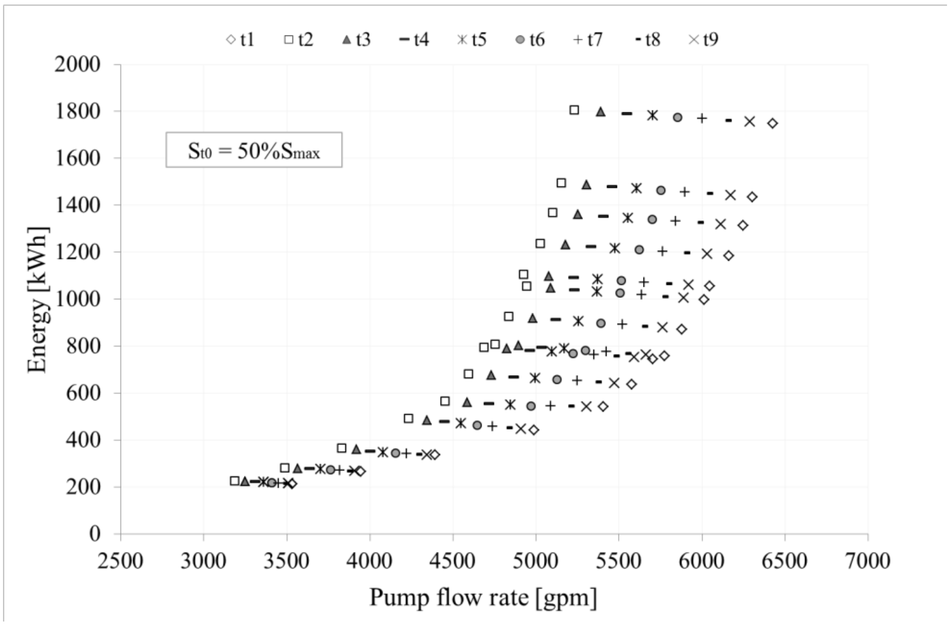

Figure 5.4 Energy consumption related to pump station flow rate for all the pump combinations and demand pattern, obtained fixing initial tank level to 50% of its maximum value.

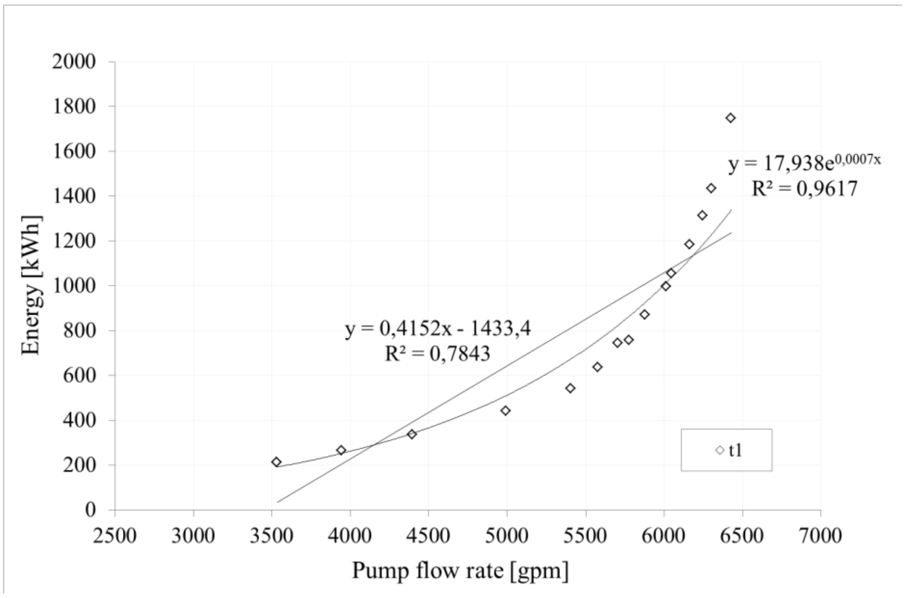

Figure 5.5 Energy curve interpolation: water use pattern t1

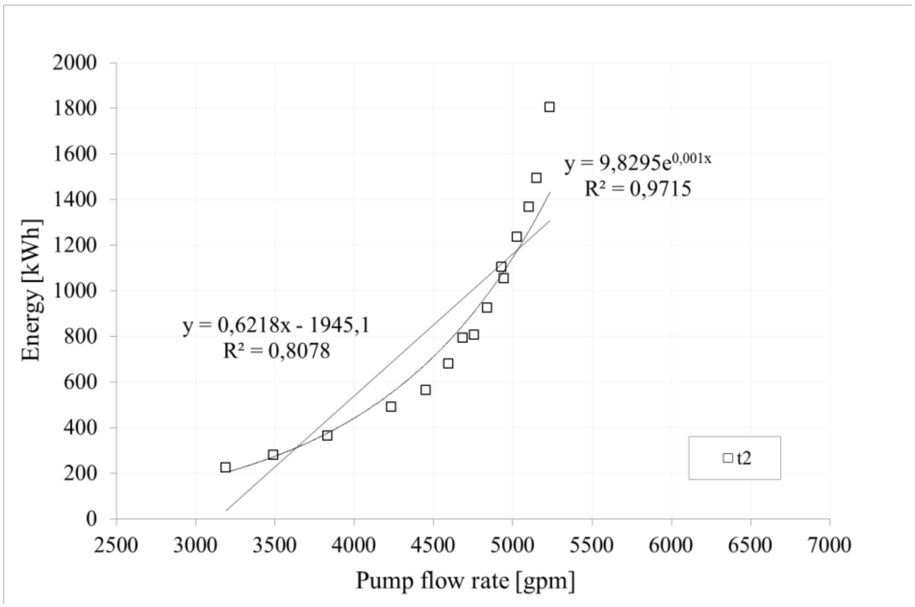

Figure 5.6 Energy curve interpolation: water use pattern t2

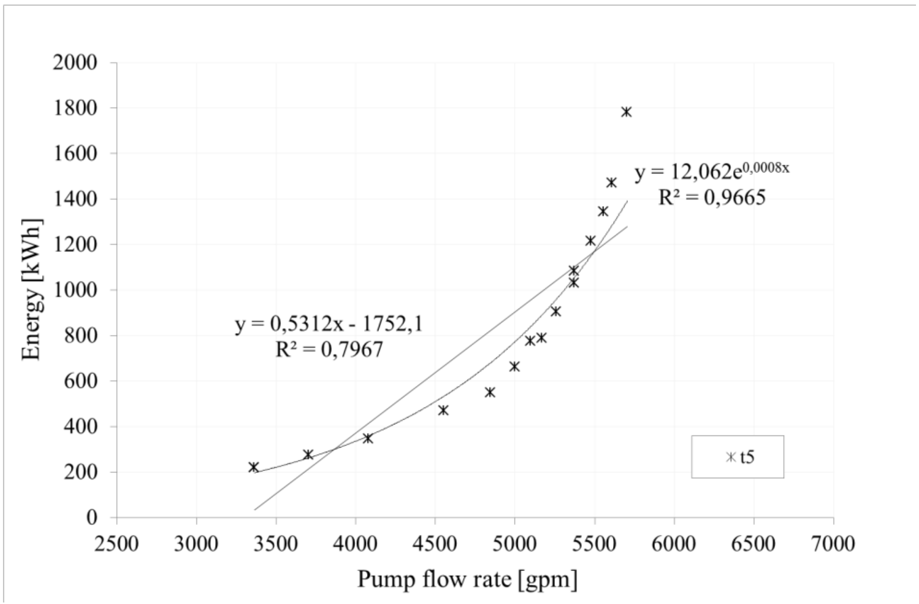

Figure 5.7 Energy curve interpolation: water use pattern t5

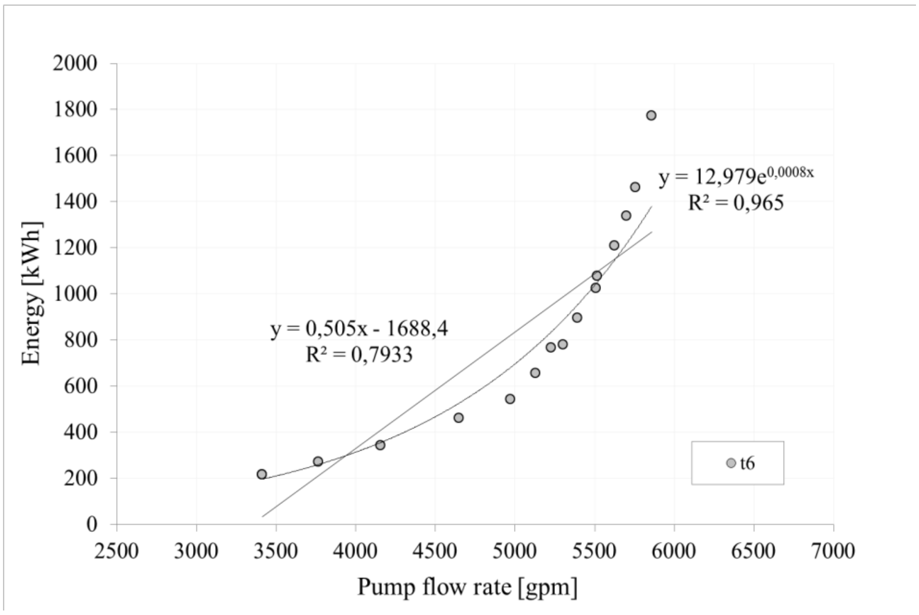

Figure 5.8 Energy curve interpolation: water use pattern t6

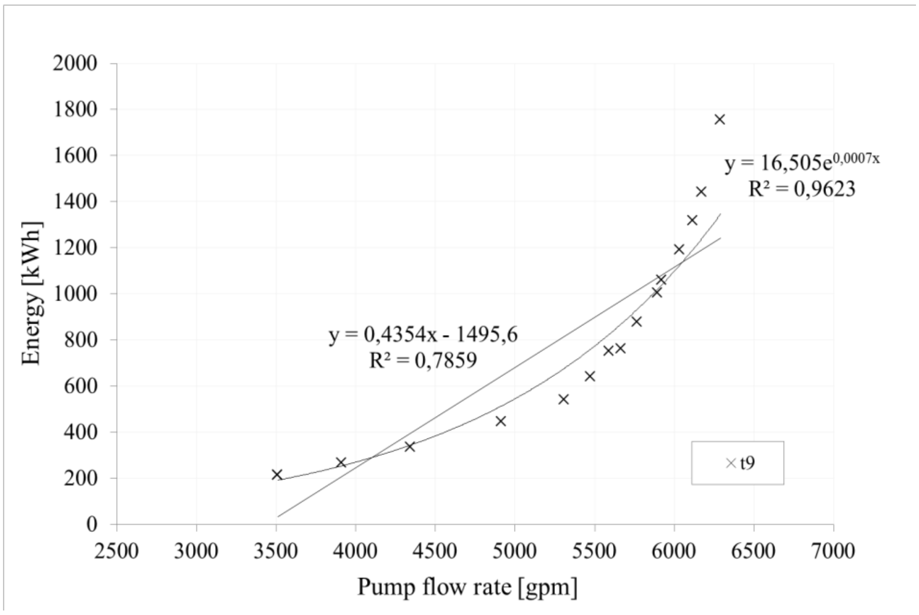

Figure 5.9 Energy curve interpolation: water use pattern t9

The LP solution was quickly identified once all of the information about constraints and objective function coefficients were calculated. At this stage the continuous solution has to be converted into a discrete schedule. To this aim, a *connection matrix*, which relates pump discharges, user water demand and pump combinations, resulting from steady-state simulations already used for the objective function coefficient evaluation, was defined. For each control interval the most suitable schedule was accomplished by selecting the combination able to provide the required discharge with lower energy consumption. The discharges do not match exactly the solution generated by LP, hence it can be chosen among either the closest, or the nearby higher value. In principle, the combination selected according to the second approach, should always allow to individuate a feasible solution, but it will be more expensive. The *connection matrix* can be updated whenever new data are available.

Figure 5.10 and Figure 5.11 show the pump station discharge and tank water levels during the optimization period, respectively; both the LP model solution and the EPS results related to the derived-schedule are presented. Moreover, the user demand profile and the energy tariff are shown.

The tank is filled during the low-cost period while it is allowed to almost empty (minimum level: 7ft) during the more expensive daytime tariff, at which point the pump station supplies nearly the entire users' demand. The cost associated with the derived-schedule is about £377/day.

The water discharges related to this derived-schedule must be considered as hourly average values: since the minimum value of pump flow rates resulting from the possible combinations (see Figure 5.4) was higher than the most of the values of the LP solution obtained. The pump combination (except the "no-pumps running" combination) able to provide the closer flow rate was chosen and then forced to work for less than one hour in order to ensure the mass balance within each LP control interval with a reasonable tolerance (about 10%). Conversely, maintaining the pump state as "on" during the entire control interval can result in an overflow in the tank. For the analysed case study the LP 24-hour solution was transformed into a schedule with a 15-minutes interval in order to verify the mass-balance constraint fulfilment.

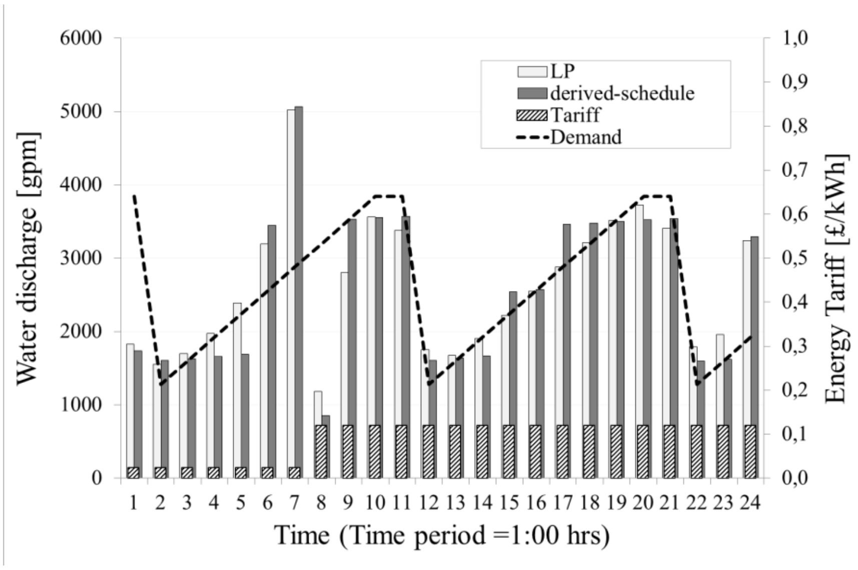

Figure 5.10 The pump station discharges from the LP model and from the simulation of the derived schedule, energy tariff and user demand for all optimization control intervals.

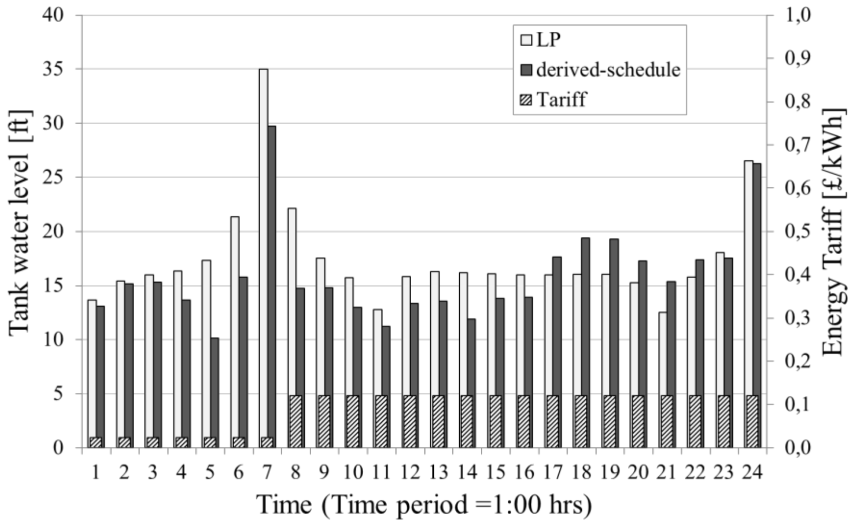

Figure 5.11 The tank water level from the LP model and from the simulation of the derived schedule, and user demand for all optimization control intervals.

The LP ability to find an optimal solution by varying the boundary conditions was also investigated. The initial water level in the tank at the beginning of the optimization is also the water level that is achieved at the end of the period. Thus, changing the initial water level does not alter the total amount of water that has to be pumped, but the times at which the water must be pumped.

One hundred different cases were considered by randomly altering the initial water level between the minimum and the maximum tank water level. The costs associated to this different optimization problems are reported in Figure 5.12. The schedule was selected by considering the discharge value closer to the LP continuous solution. This approach provides feasible solution as long as the level is lower than $0.76S_{max}$. The pumping cost follows a piece-wise linear function, four different tank initial water level interval can be individuated; for each of these intervals the cost remains the same. The pumping cost increases with the tank initial level as long as the level reaches the $0.69S_{max}$, then the cost decreases. Such trend can be explained by the fact that, although free water is stored into the tank at the beginning of the simulation period, and then much more water volume is available to satisfy the demand, other parameters, such as the water use pattern and the electricity tariff variability during the control period, influence the optimal solution.

Moreover, the trend showed in Figure 5.12 should be conditioned by the schedule selection process. Figure 5.13 shows the results of the same analysis in which the pump scheduling selection is guided by a different approach. The schedule is identified considering the combination which provides the nearby higher pump discharge respect the LP solution. For all the tank initial water levels tested, this approach leads to feasible solutions, but the corresponding operational cost is higher than the previous one. Moreover, the overall trend is quite different, the lowest cost is reached around the $0.4S_{max}$, while both for lower and higher values, the energy consumed is greater, albeit slightly.

Therefore, the schedule identification revealed some criticality due to the appropriate selection of the pump combination for each control interval. Choosing the closest value leads to find less expensive solutions, but the feasibility is not guaranteed for any boundary conditions. Hence, if the feasibility is a stringent requirement, such as when LP model works in stand-alone application, the schedule, corresponding to the nearby higher value of pump discharges, can be selected.

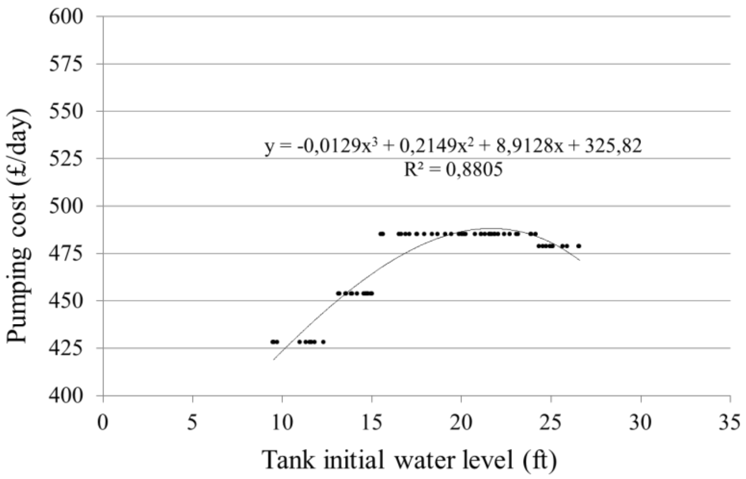

Figure 5.12 Pumping cost resulted from different LP problems obtained by changing tank initial water level, the schedule is selected by choosing the closest value of discharge.

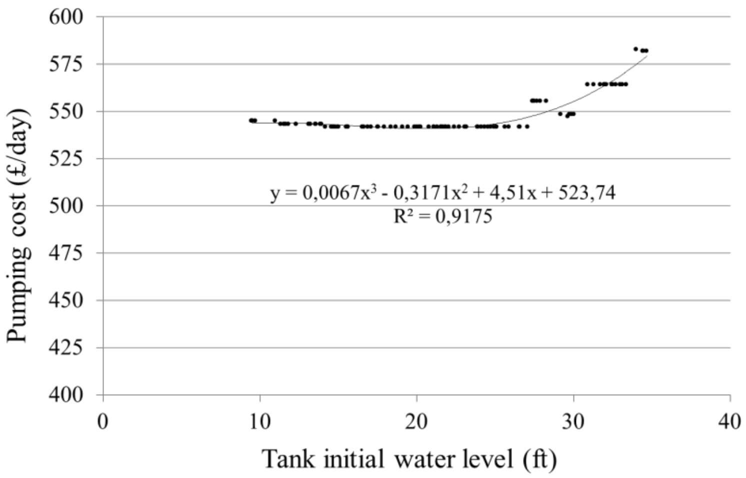

Figure 5.13 Pumping cost resulted from different LP problems obtained by changing tank initial water level, the schedule is selected by choosing the nearby higher value of discharge.

In order to test the suitability of the LP model to solve optimization problem for complex networks, among those proposed in literature, the “three pressure zone” water distribution network presented by Price and Ostfeld (2013b) was chosen.

This network, described in section 4.4.2, poses the problem of describing the mass balance equations for a multi-tank system. The mass balance equation has to take into account the connections between the sub-networks and the different water use patterns. Moreover, the presence of four pumping stations and an optimization period on 168-hours basis, increases the number of the decision variables until 672 for each of the monthly optimization problems.

Each pumping station comprises only one pump. In order to define the relation between energy and pump flow rates, several steady-state network simulations were carried out for all the pump combinations and water use demand, by setting the initial water level in the network tanks to 50% of their maximum value. Once the objectives function coefficients were calculated, the LP problem was solved. The continuous solution was then transformed into a discrete one identifying the optimal schedule for each week. The cost associated to this schedule was reported on monthly basis in Figure 5.14 and compared with the “all pump running” solution. The best improvement respect to this baseline is the 43.5% of cost reduction in January, while the worst (about 20%) is related to November and April. Moreover, the operational cost reduction reaches averagely the 34% in the summer time (July – Sept).

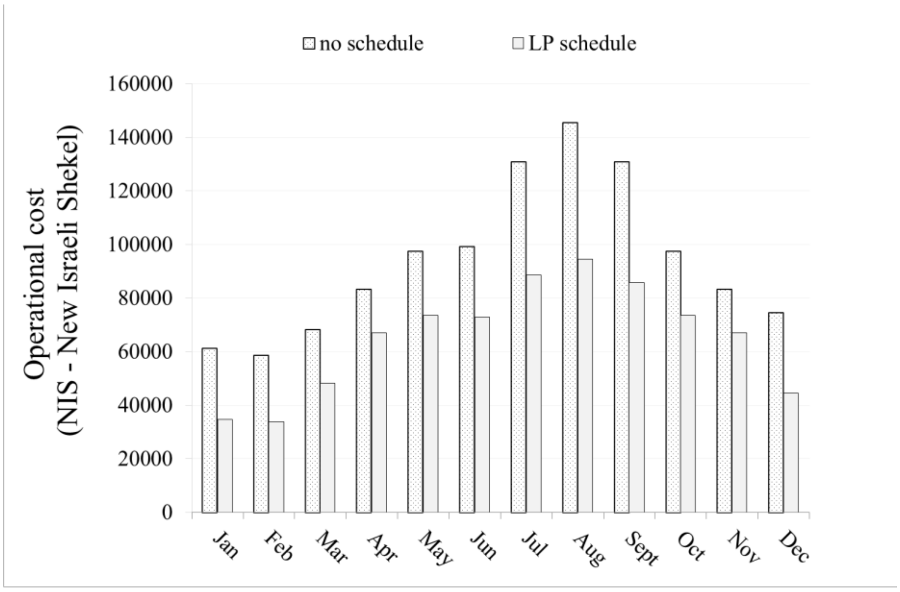

Figure 5.14 Monthly operational cost resulted from the "all pumping on" and LP-derived schedule.

The results were also compared with the operational costs provided by Price and Ostfeld for the *case 1*, in which their model was solved including water balance and hydraulic headloss constraints. The monthly operational cost are not explicitly reported in the referred reference, but they were kindly extended by the authors after a request.

The model proposed by Price and Ostfeld has lower operating costs for all months except for August (Figure 5.15). Probably, the boundary conditions and the energy tariff in this month do not allow further optimization, then both the methods reach the same result. The distance between the solution provided by the LP model and that by Price and Ostfeld model vary with the considered month. It is not possible to have the same trend due to the different boundary conditions (e.g. water use pattern) and electricity tariff (each month presents a different distribution of the electricity tariff, which is divided in low, moderate and peak charge).

On annual basis, the average cost reduction provided by the Price and Ostfeld model, respect to the baseline "all pump running", is about 49.3%. Whereas the LP model, herein proposed, produces a yearly cost reduction of about 31%.

In Figure 5.16 the operational cost per cubic meter of user demand corresponding to the baseline schedule, the Price and Ostfeld schedule and the LP-derived schedule is presented. The average values are 0.20, 0.10, 0.14 NIS/m^3, respectively.

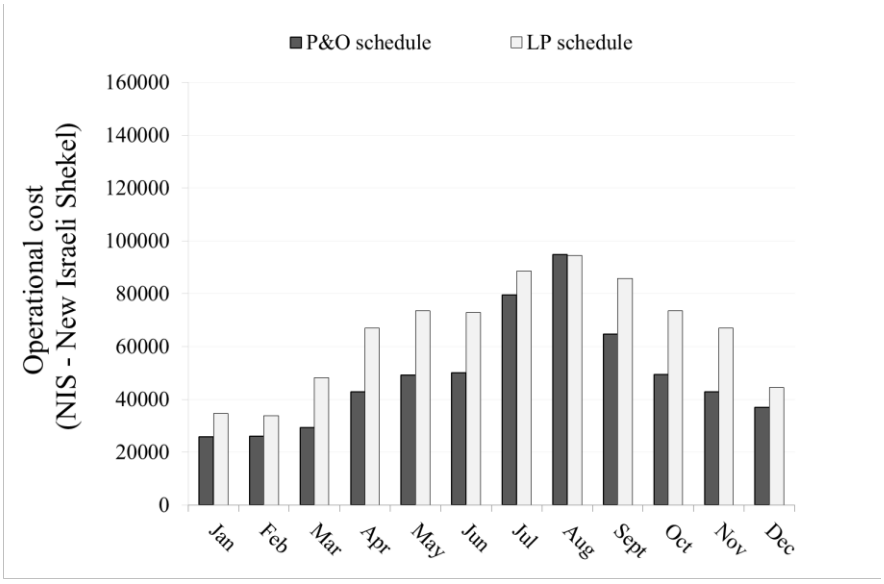

Figure 5.15 Monthly operational cost resulted from the LP-derived schedule and Price and Ostfeld schedule – *case 1*.

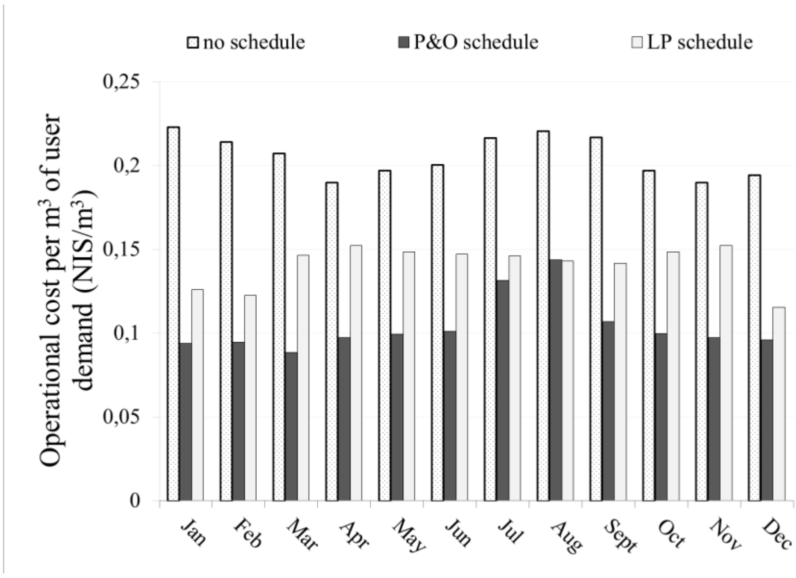

Figure 5.16 Monthly operational cost per m^3 of user demand related to no schedule, Price and Ostfeld solution – case 1 and LP-derived schedule

The LP must be able to cope with changes in the optimization problem. In particular, as already shown for the Anytown network, the initial tank water levels were altered at the beginning of the optimization period in order to verify the LP suitability on finding an optimal solution. The initial water level for each network tanks was assigned according to an uniformly random distribution, a set of 100 values were tested. For each month the optimal operational cost was identified; the cost do no vary consistently among the simulations. In January, for example, the average cost is 34676 NIS, with a variance of 0.75. This result is confirmed by the 25th 50th and 75th percentiles which are equal to 34673.5 NIS, 34676.4 NIS and 34679.2 NIS, respectively. While with regard to June, the average value is 73761 NIS, and the percentiles are 73717.7 NIS, 73721 NIS and 73724.9 NIS. Moreover, the results showed that there is no correlation between operational cost and initial tank water levels.

5.2 LP model reliability

In order to evaluate the reliability of the LP, a comparison was made with solutions generated by the evolutionary algorithms. Such analysis was carried out with regard to the benchmark Anytown network.

Firstly, twelve different optimization runs were performed from different, randomly selected, starting points with the Hybrid Discrete Dynamically Dimensioned Search (HD-DDS) algorithm. The algorithm runs with a 1-hour interval, for this reason the previous schedule, obtained by solving the LP model, was transformed accordingly; it was simply assumed that the same pump combination working for one hour, even when it was forced to run for less (15 minutes-interval); the cost associated to this LP-derived schedule is £479. This value is not far from the results obtained by HD-DDS, of which the average value is £407, as shown in Table 5.1.

Table 5.1 The best solutions obtained from HD-DDS different initial seeds.

Best solution (£/day)			
Run number	HD DDS with randomized initial solution	Run number	HD DDS with randomized initial solution
1	408.04	7	387.51
2	417.44	8	414.71
3	405.26	9	416.88
4	406.72	10	404.06
5	401.17	11	422.20
6	381.11	12	420.52
		Average	407.135

The LP model optimal solution was also compared with the pumping cost resulting from six single-objective genetic algorithm optimization runs with a randomized initialization. The number of population was set to 150, while the generations to 1000. The mutation and the crossover coefficients are fixed to 1.5 and 2, respectively.

The obtained average value is about £402. As well as the previous comparison, the LP solution is quite promising (Table 5.2).

Table 5.2 The best solutions obtained from GA different initial seeds.

Best solution (£/day)	
Run number	GA with randomized initial solution
1	399.34
2	393.73
3	406.08
4	394.44
5	416.43
6	400.27
Average	401.715

5.3 Hybrid optimization model

As above mentioned, the LP-derived schedule was tested as initial seed solution in both HD-DDS and GA in order to investigate the hybridization capabilities.

Table 5.3 shows the solutions generated by the HD-DDS algorithm testing the LP-derived schedule as initial seed solution. The cost associated with the results derived from the LP schedule initial solution varied between £373.14 and £404.05 with an average of £383.91. The values are lower than those obtained running the HD-DDS with the randomized initialization (Table 5.1). The best improvement is in run 12 where the LP derived-schedule initial solution improved the HD-DDS solution by 10.7%. Therefore, it is interesting to notice that the cost of the schedule with a 15-minutes interval is very close at £377. In Figure 5.17 the solutions generated by HD-DDS algorithm both with LP-derived schedule and randomized solution as initial seed are shown.

Table 5.3 Solutions generated by HD-DDS algorithm with LP-derived schedule as initial solution.

Best solution (£/day)			
Run number	HD DDS with LP initial solution	Run number	HD DDS with LP initial solution
1	384,92	7	381,80
2	376,48	8	387,27
3	396,02	9	379,67
4	380,26	10	382,98
5	384,88	11	404,05
6	373,14	12	375,42
		Average	383,91

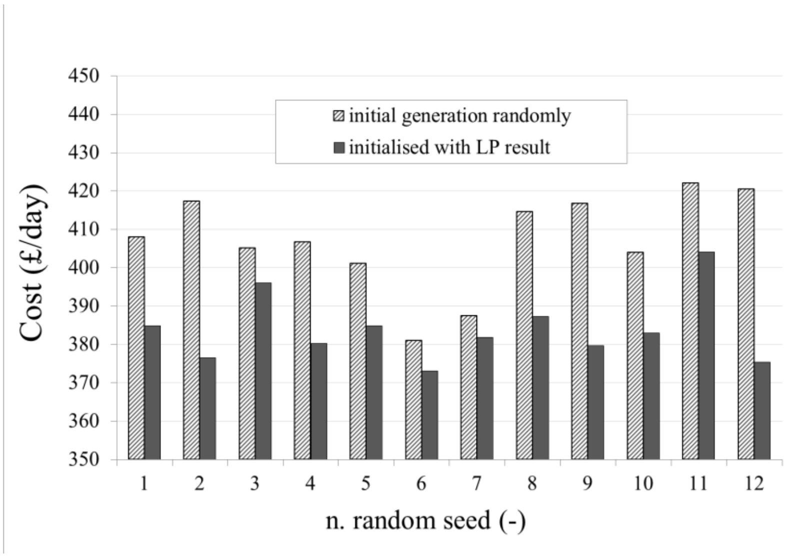

Figure 5.17 Comparison between solutions generated by HD-DDS algorithm with LP-derived and randomized solution as initial seed.

The LP capability to improve the convergence of GA was also investigated. In Table 5.4 the solutions obtained from six different optimization runs using the LP-derived solutions as initial seed are reported. For all the optimization runs, the GA provides lower costs respect to those generate with a random initialization (Figure 5.18). The best improvement is in run 5: equal to 10.6%.

Table 5.4 Solutions generated by GA with LP-derived schedules as initial solution.

Best solution (£/day)	
Run number	GA with LP initial solution
1	367,88
2	367,88
3	368,60
4	368,60
5	372,30
6	367,88
Average	368,86

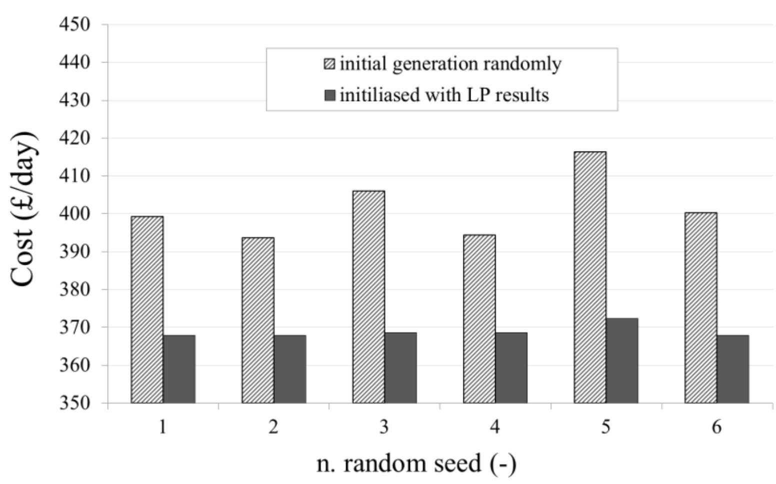

Figure 5.18 Comparison between solutions generated by GA with LP-derived and randomized solutions as initial seed.

Further considerations can be drawn by the objective function (pump operating cost) evaluations during the GA optimization respect to the different seeding.

Figure 5.19 shows the pumping cost evaluations with regard to the random seed 2, for both randomized and LP-derived initialization. Less than 16 iterations are sufficient to the hybrid model for identifying a better solution respect to the randomly initialized GA. In this case, the best solution is reached after more than 800 evaluations. In other words, the application of the LP initialization leads to obtain the same solution generated by traditional GA by 25 seconds rather than 1200 seconds.

Moreover, with regard to random seed 5, in which the hybrid model provides the best improvement respect to the traditional GA, less than 10 iterations are suitable to identify lower pumping cost (Figure 5.20). In this case the traditional GA needs about 240 seconds rather than 15 seconds of the hybrid model.

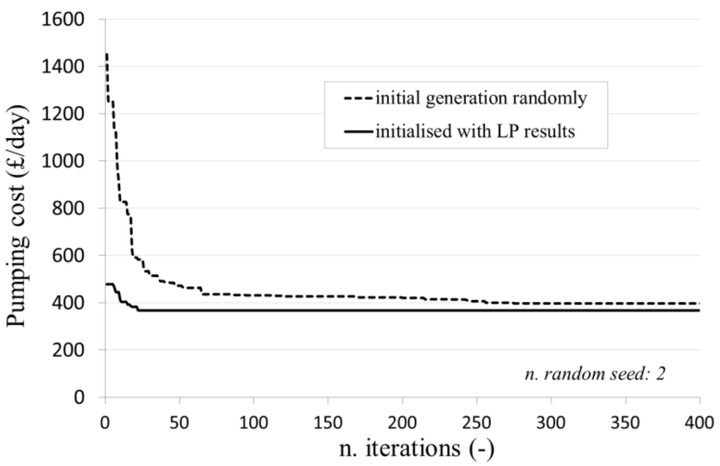

Figure 5.19 Pumping cost evaluations during GA optimization for run 2 with random and LP-derived solutions as initial seed.

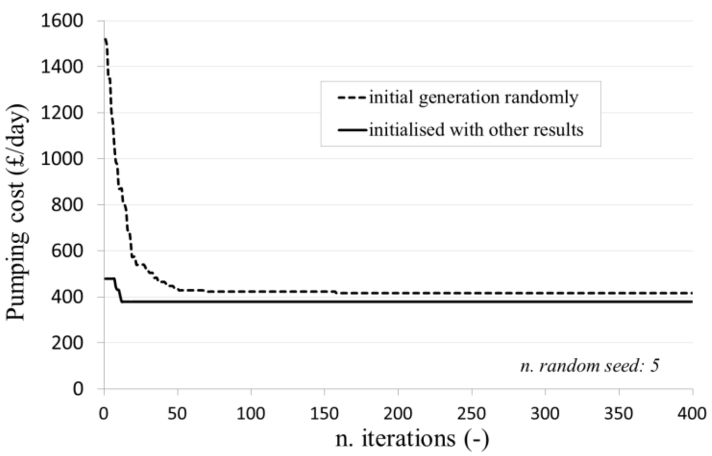

Figure 5.20 Pumping cost evaluations during GA optimization for run 5 with random and LP-derived solutions as initial seed.

Chapter 6.

Conclusions

The importance of building an optimal control system (OCS) is stated by several authors. It is recognised to be a cost-effective and efficient tool to manage water distribution systems. The complexity of such systems requires, beyond the operators expertise, a tool able to describe the hydraulic network behaviour and to generate operational solution able to reduce cost, but in the same time, satisfying the system constraints. The OCS can be realized in Real-Time Control (RTC) coupled with SCADA system. Hence, the major requirements have to be robustness, speed, reliability, accuracy and confidence.

As mentioned above the OCS comprises three components: hydraulic, demand forecast and optimization model.

Having a well-calibrated network model and an algorithm able to simulate different operating configurations, plays an important role in the water distribution system analysis. The current software do not allow to describe properly some operating conditions, such as those put into effect during intermittent supply. For this reason, one of the objectives of this thesis was the development of a hydraulic solver which implements, as well as the traditional demand driven approach, the head driven one, focusing on a novel relationship, able to take into account the presence of local tanks along the network.

The consumers install private tanks in order to cope with intermittent water distribution, by collecting water during service period to use it when the service is not available. This peculiar network configuration leads the system to work very far from the design assumptions. Moreover, even when the supply becomes continuous, consumers maintain the private tanks due to the lack of confidence on the water utility managers.

Such kind of configuration is not efficient from energy point of view, both for the users and the water utilities, which have to face with different objectives: providing a good level of services, while keeping under control water losses and energy consumptions.

The network solver was used to test different solution for the water losses and energy management in real water distribution networks. Moreover, it was applied to verify the feasibility of the optimal solutions generated by the optimization model.

The optimization model was defined to reduce the cost associated to the water distribution system operation. As pointed out in the technical literature, the major operational cost is the pumping energy cost. Without making changes to the basic elements of water supply system, remarkable reduction in operation cost can be achieved by optimizing the pump scheduling problem. This problem has to be solved quickly with an acceptable reliability in order to be effective.

For this reason, in this thesis the pump scheduling problem was investigated through a Linear Programming based method. Once the constraints and objective function coefficients were formulated, the decision variables – the pump station water discharges – were quickly obtained solving the linear system of equations which describes the above-mentioned optimization problem. Then, the LP solution was transformed in a discrete schedule able to provide the same rate on a 24-hour basis. This step of the methodology revealed some criticality due to the appropriate selection of the pump combination for each control interval. To overcome this problem the schedule can be evaluated by reducing the size of the control intervals or by changing the conversion approach. In particular, the different discretization ensures that the mass balance constraint is respected, this approach has proved to be a good solution for the tested case studies. Whereas, converting schedule by following the approach for which the selected combination is that able to provide the nearby higher discharge respect to the LP solutions, generates always a feasible solution but with higher cost.

The LP methodology demonstrates, therefore, the potential to be adopted to rapidly determine an approximate, though acceptable, solution which may itself then be subject to further optimization. When the schedule with an hourly interval was tested as an initial seed solution into the evolutionary algorithms (Hybrid Discrete Dynamically Dimensioned Search, HD-DDS and genetic algorithm, GA), the resulting cost was lower than that obtained using the randomized initialization for several different optimization runs. With regard to GA optimization, the convergence improvement was also demonstrated in term of simulation time. For the best optimal solution, the hybrid algorithm has shown to reduce the computational efforts by 16 times.

The steps which can be considered time consuming into the methodology above presented, are the evaluation of the objective function coefficients. Nevertheless, this limitation should be overcome running the hydraulic model off-line and collecting data which can be adopted for similar conditions.

The methodology herein presented can be easily introduced into a general scheme of operation of an optimal control system (OCS) for real time control. In particular a feedback/feed-forward control scheme is proposed.

Figure 6.1 shows a real time OCS framework for the water distribution system (WDS). In particular, the sensors installed along network monitor state variables with a specific time resolution. These data are stored into database and transmitted to the OCS, directly, in case of data characterized by adequate quality, or after a pre-processing stage aimed to models adaption. Afterwards, the demand forecast model predicts water demand. The water distribution system model determines the network behaviour respect to different operational strategies generated by the optimizer, considering the estimated water demand as boundary condition. The optimizer (or the optimization model) creates optimal control strategy by minimizing the cost function while the operational and hydraulic constraints are satisfied. Within the optimizer the selection of the algorithm (e.g. stand-alone LP or hybrid model) is dependent on the boundary conditions and on timing to put into effect the operational strategies.

The optimal solution is then used to set controllers, which regulate actuators to fulfil the desired value with a reasonable tolerance. The sensors register also the variation on the process variables, which can be used to improve the OCS output. The different parts of the control network are connected by means of the transmission system that convey data between them. The data collected by sensors are stored in database. Moreover, during the optimization period, data exchange is expected between OCS and database in order to reduce e.g. the simulation time when similar boundary conditions or objectives occur.

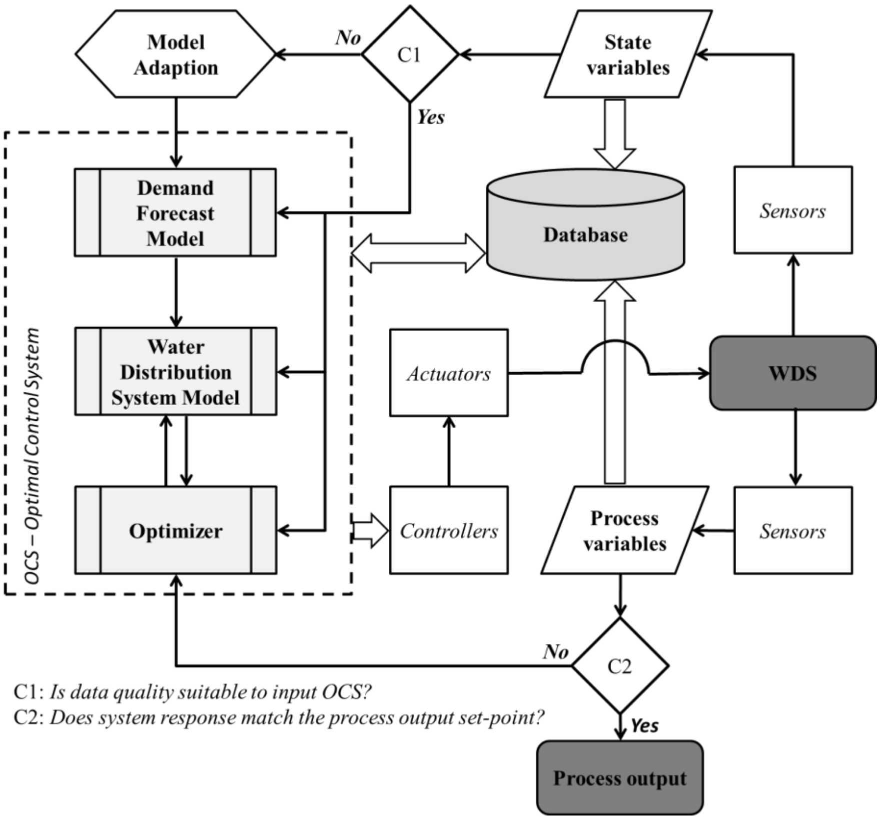

Figure 6.1 Real Time Optimal Control System framework.

6.1 Further development of research

Further studies are needed in order to verify the reliability of the proposed methodology to real-system applications, at least with regard to optimization model. A more efficient selection of the pump combinations may be performed both for the objective function coefficients evaluation and the selection of the pump schedule. Moreover, the automatic detection of the network connections may be introduced in order to develop quickly the optimization constraints.

Finally, the proposed optimal control framework can be improved by taking into account a multi-objective optimization approach and integrating a demand forecast model.

APPENDIX A

Further applications of the hydraulic network solver

A.1 Introduction

As mentioned in section 3.4 the hydraulic network solver was applied to carried out different analyses with regard to water losses and energy recovery in water distribution systems. In the following, two case studies and three different applications are presented. Namely, a real losses analysis was carried out with regard to the sub-network 2 Oreto-Stazione (Palermo, Italy); while the District Meter Area (DMA) of the sub-network Noce-Uditore (Palermo, Italy) was involved for the investigation of the network pressure influence on apparent losses and the possible energy recovery by means of Pumps as Turbines (PATs) installation.

A.2 Case studies

A.2.1 Sub-network 2: Oreto-Stazione

The entire water distribution system of the city of Palermo (Italy) is made of 17 sub-networks that supply as many zones of the city. One of these sub-networks (Figure A.1) was chosen as case study because all its geometric characteristics are precisely known, as well as the number and the distribution of user connections, the water volumes delivered and measured, and the pressure and the flow values in a few important nodes (Figure A.1).

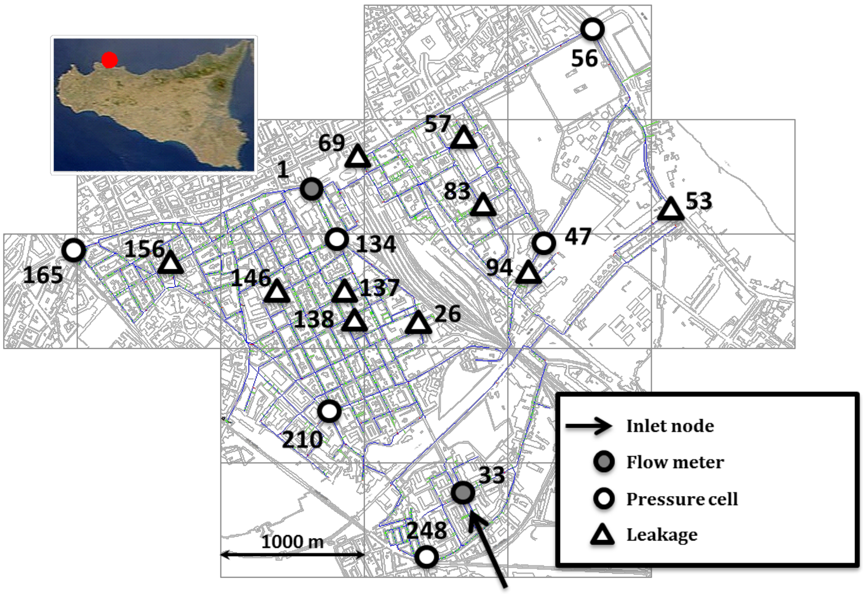

Figure A.1 Schematic of the sub-network Oreto-Stazione.

This network was, in fact, totally rebuilt in the 2002, even if the renovation process regarded the sole distribution network keeping in service the old cast iron feeding pipes that connect two reservoirs at different levels to the network. The two reservoirs can store up to 40,000 m^3 per day, and supply around 35,000 inhabitants (8700 users).

The entire network is made of polyethylene and it is about 40 km long. The pipes have diameters ranging between 110 and 225 mm. The network was designed to supply about 400 l/capita/d, but actual average consumption is about 260 l/capita/d. As consequence, the network was characterised by low water velocities and correspondently high pressures, which resulted in background leakage in the past. Intermittent distribution on a daily basis was introduced as a common practice over the last five years (at least during summer period), leading users to acquire private tanks, with specific volume equal to 200-250 l/capita. Furthermore, the water utility decided to reduce the pressure level on the network due to the significant water losses that occur in the feeding pipe. This encouraged users to maintain their own tanks.

The network is monitored by six pressure cells and two electromagnetic flow meters (Figure A.1) that have provided data on hourly basis almost continuously since 2001. The calibration of the network hydraulic models is constantly updated when new data become available.

Real losses were investigated in Autumn 2003 by insulating parts of the network and evaluating night water balance in the insulated network trunks.

Nine different leakage locations were found in the network (Figure A.1) and the average leakage flows range between 10 l/min and 60 l/min.

A.2.2 DMA: Noce-Uditore

A small DMA of the distribution network of Palermo, Italy (Figure A.2) was selected since informations about topology, hydraulic characteristic, user's plumbing systems were known.

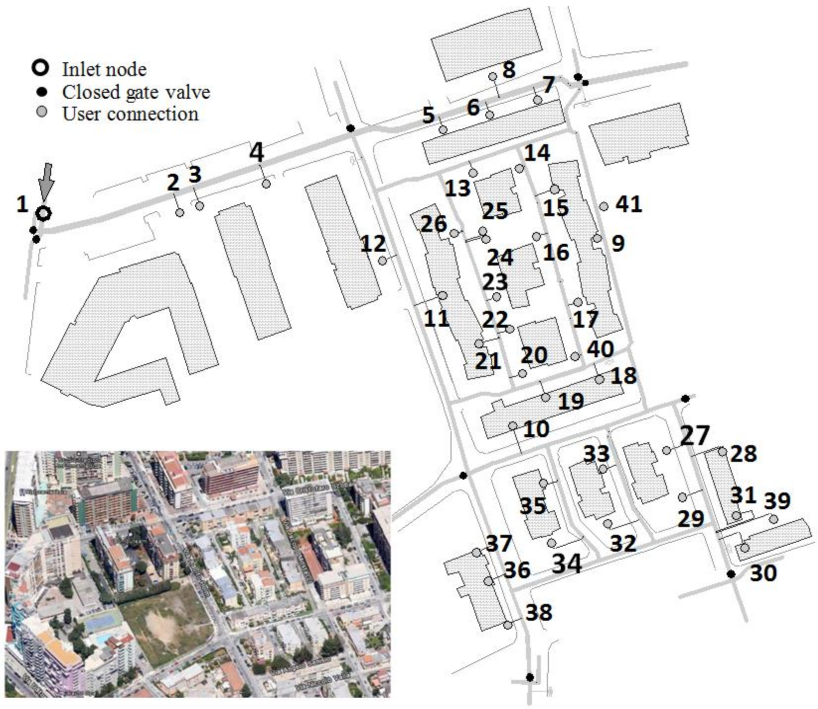

Figure A.2 Schematic of the District Metered Area

The network is about 1.3 km long, and the pipes are made of high density polyethylene, with diameters ranging from 110 to 225 mm. The network presents 40 service connections that supply a total of 164 residential end-users equipped with multi-jet water meters (Table A.1). Water meters that are more than 10 years old are in class B, according to ISO-4064 (1993) and Directive 75/33/EEC (1974); others are in class C (ISO-4064 2005).

All the end-users have previously installed private tanks downstream of the revenue meters because the water supply was intermittent in the past. Namely, 34 service connections supply single users with a private tank located on the building roof, while six service connections supply condo collective users with private tanks located under street level.

Table A.1 Water meter characteristics

Diameter [mm]	1-5 [years]	5-10 [years]	10-15 [years]	15-20 [years]	Total	%
15	80	31	33	13	157	96
25	3	1	0	0	4	2
40	1	1	0	0	2	1
50	0	1	0	0	1	1
Total	84	34	33	13	164	100
%	51	21	20	8	100	

Since the end of 2009, the DMA was monitored and the input volume and the pressure at the inlet node were measured with a temporal resolution of 30 min. An average water consumption pattern was obtained by recording the water consumption of five single users installing downstream of their private tank a new class C water meter coupled to a data logger.

Water losses due to real leakages in the DMA were evaluated through a noise logger survey and a night flow analysis, where the minimum night flow was continuously recorded and analysed. No leaks were found and the background leakage level was assumed to correspond to the minimum measured flow rate due to the residential night water demand. Prior to these measurements, a specific leak detection survey was performed in the users' plumbing system, and all detected leaks were repaired before the monitoring period started. Water theft did not occur in this district, and meter-reading errors were excluded, as each reading was verified twice by independent operators.

A.3 Leakages modelling

This research was carried out with regard to the sub-network Oreto-Stazione. The model was initially calibrated and validated in order to evaluate its reliability. According to the applied approaches, the main calibration parameter is the pipe roughness in the head losses formulation. The model was calibrated according to the pressure profiles available in the six pressure gauges present in the network (Figure A.1). The calibration process showed that pipes work mainly in the hydraulically smooth regime so the model showed to be rather insensitive to roughness in the range ($0.01 < ks < 0.001$ mm) considered in the present study. For this reason, the roughness k_s was arbitrarily set equal to 0.01 mm. To perform the pressure driven analysis it was needed to fix the minimum head required to have outflow at the node equal to the tank elevation at each node and the pressure head required to satisfy the demand which is assumed to be equal to +10m considering the head losses in the user plumbing. The general formulation of the leakage-discharges was applied (see section 3.2.2).

The leakage model parameter *a* was obtained by data interpolation of field monitoring, while for the *b* parameter the value was fixed equal to 0.8 according to previous study for PEAD pipes (Ferrante et al. 2010). Actually this value is not a constant depending on the leak size and shape and on the pressure head time history and rate variation.

The steady-state model was compared with the results of a dynamic approach, of which details are reported in De Marchis et al. (2010). The maximum difference between the two models is most likely to be found during intermittent distribution when the dynamic behaviour of flows in the network should be more evident. For this reason the comparison was carried out during the first 24 hours after the start-up of the network, initially assumed to be empty. The steady state model assumes that the network fills up instantaneously while the dynamic model is able to analyse the evolution of the network pressures during filling process. Figures A.3 – A.6 show leakage flow from 4 reported losses in the network, respectively. The analysis of the figures shows that, after network filling (after about 100 minutes), the two models do not show relevant differences (between 3% and 5%) most likely due to the small differences in pipe roughness calibration. A large difference is indeed visible during network filling: the steady state model greatly overestimates leakages because network pressures are largely overestimated.

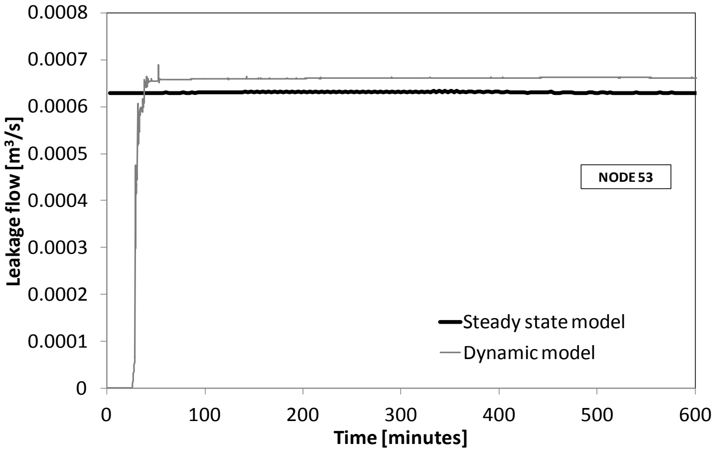

Figure A.3 Comparison between dynamic and steady state model in the evaluation of leakage flow in node 53.

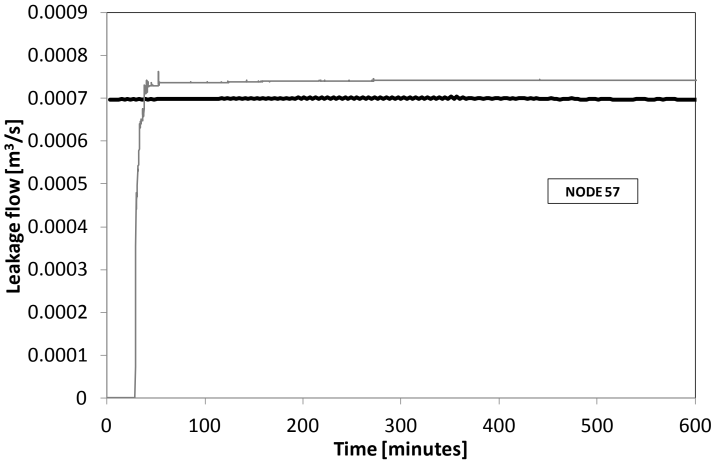

Figure A.4 Comparison between dynamic and steady state model in the evaluation of leakage flow in node 57.

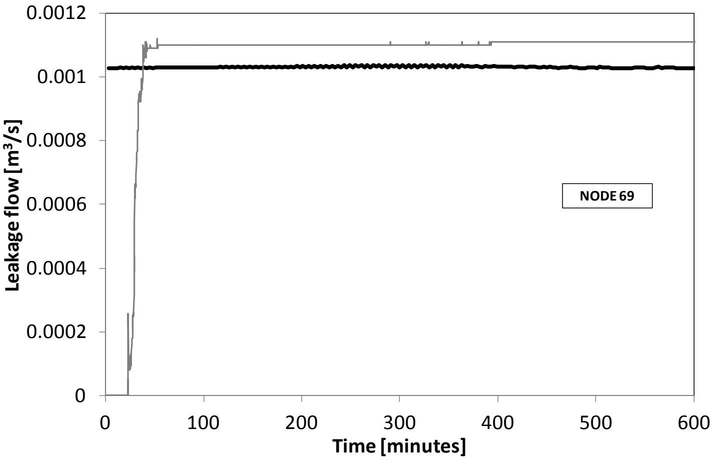

Figure A.5 Comparison between dynamic and steady state model in the evaluation of leakage flow in node 69.

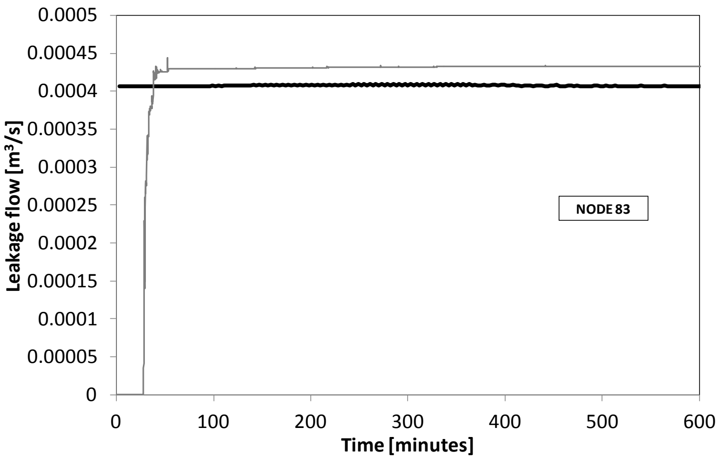

Figure A.6 Comparison between dynamic and steady state model in the evaluation of leakage flow in node 83.

Considering the global loss volume, Figure A.7 shows the differences between the first 100 minutes (Figure A.7a) and the whole simulated day (Figure A.7b). The steady state model overestimates the leakage volume of about 33% during the network filling process. This overestimation is compensated during the day by the slight overestimation of the leakage volume by the dynamic model. After 24 hours, the steady state model underestimates the leakage volume of about 7%.

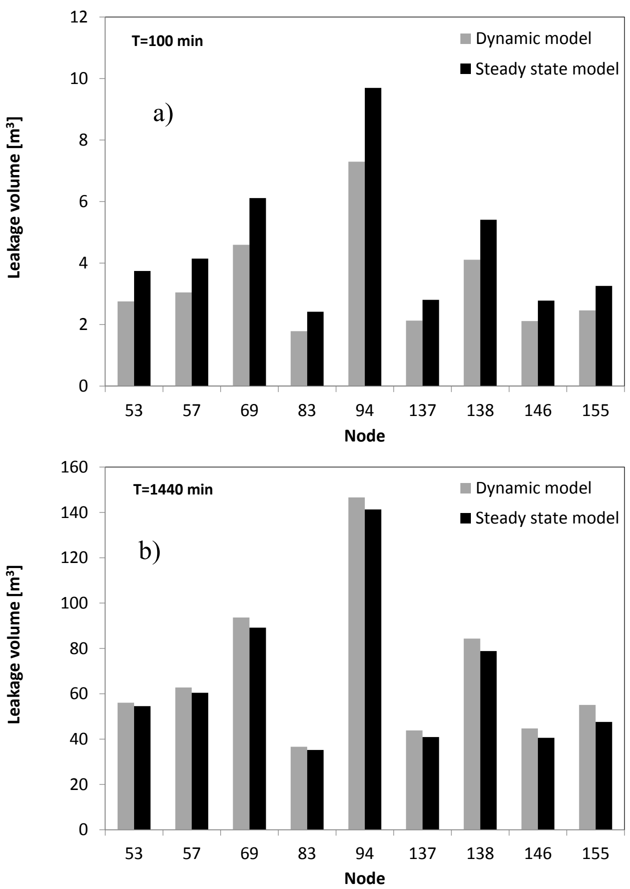

Figure A.7 Comparison of average leakage volume after the first 100 minutes of simulation (a) and after 1440 minutes (b)

A.4 Apparent losses analysis

A specific apparent losses module was developed able to quantify the intrinsic meter error related to the meter age, *age*, the flow rate passing through the meter, q_{act}, and the pressure at the node, P. The complete mathematical formulation of the model and the experimental campaign used to develop it are provided by Criminisi et al. (2009) and Fontanazza et al. (2010a). In the present paragraph, the essential equations are reported with the related parameters. The estimation of metering error was based on the following equation:

$$q^k_{means,i} = f\left(q^k_{act,i}\right) = \begin{cases} 0 & if\ q^k_{act,i} < q^k_{start,i} \\ q^k_{act,i}\left[1 - \left(\frac{q^k_{start,i}}{q^k_{act,i}}\right)^{\gamma^k_i} \cdot \cos\left(\pi \frac{q^k_{act,i} - q^k_{start,i}}{Per^k_i}\right)\right] & if\ q^k_{act,i} \geq q^k_{start,i} \end{cases}$$

where q_{means} is the flow measured by the meter installed upstream the *k*-th tank connected to the *i*-th node [l/h]; q_{start} is the meter starting flow [l/h], defined as the flow that generates motion in the meter when the mechanism is at rest; γ is a dimensionless coefficient that takes into account the reduction of metering error with the actual flow rate passing through the meter q_{act}. *Per* is the semi-period of measurement error oscillation around zero, which accounts for both negative and positive errors depending on passing water flow [l/h].

Namely, the above equation is an empirical equation formulated by determining the best fit to the experimental error curves obtained for meters of different ages and for four different test pressures (0.5, 1.0, 1.5, 2.0 bar) (Fontanazza et al. 2010a). Those laboratory results revealed that the influence of pressure is negligible on shape parameters γ and *Per* whilst it is relevant on q_{start}. The relationship between average starting flow and meter age, under four different test pressures, obtained by interpolating experimental results with an exponential law is also given by:

$$q^k_{start,i} = a_1 \cdot e^{a_2 \cdot age^k_i}$$

Table A.2 shows a_1 and a_2 values related to the four different test pressures; parameter values for intermediate pressures were obtained by linear interpolation (Fontanazza et al. 2010a).

Table A.2 Parameters of relationship between average starting flow and meter age, under different test pressures

Pressure [bar]	a_1	a_2
0.5	4.135	0.045
1.0	3.717	0.046
1.5	3.373	0.047
2.0	3.015	0.050

In distribution networks pressure reduction valves (PRVs) are often used by water utilities to control the pressure and reduce background losses. These practices could influence the performance of water meters. For this reason, a PRV model (section 3.3.2.1) was implemented and integrated with the demand

and the hydraulic network models (see section 3.3.3) to better estimate the effect of pressure management on apparent losses. The proposed mathematical model is applied to the Noce-Uditore DMA.

The hydraulic network model was not calibrated with regard to the pipe roughness because pressure data were available only in the DMA inlet node. The absolute roughness was set to 0.02 mm. The node head-discharge model parameters were set to the averages obtained in a field campaign that was carried out in 2007 and 2008 (Criminisi et al. 2009): C_v was set equal to 0.57 and a_v to 2.8 cm^2 for each tank. The elevation and the dimension of the tanks were measured in the same monitoring campaign. P_s was set 20 m higher than the float valve opening and P_{min} was set equal to the float valve height.

Finally, the PRV model parameters were chosen as follows: the needle valve speed control setting, was set equal to a constant value, 10^{-6}; the value of the PRV set point, was chosen equal to 17 m to guarantee the tank filling according to the user's tank position (on the rooftop or underground); and the characteristic curve was selected according to Prescott and Ulaniki (2003):

$$C_{prv} = 451877x_m^4 - 29538x_m^3 + 56696x_m^2 - 0.417x_m$$

The comprehensive model (hydraulic simulation network, demand and apparent losses models) was applied to the case study and apparent losses at each node were evaluated as the sum of the metering errors of all the connected meters (Figure A.8). The apparent losses, i.e. the difference between the volume measured by the meter and the actual consumed volume, was normalised by the actual consumed volume and expressed in percentage. In order to summarise the results for each node, Figure A.8 shows the median daily losses (the black cross), the 25th and 75th quantiles (the end and the top of the box), representing respectively, the apparent loss that is not exceeded for 25% of the day and the apparent loss that is not exceeded for 75% of the day, the 5th and the 95th quantiles (the ends of the two whiskers) representing respectively, the apparent loss that is not exceeded for 5% of the day and the apparent loss that is not exceeded for 95% of the day. Nodes are ordered against average water meter age and they were divided in three age groups.

Using node 33 as an example, Figure A.8 shows that the median apparent loss is 12% and users were supplied for 6 hours (sparse during the day) with high flows that guarantee very low apparent losses (below 2%); only for the 5% of the day (a little bit more than one hour), apparent losses are higher than 23%. Looking at node 13, the level of apparent losses is much higher: the median loss is around 38%, apparent losses are lower than 10% only for 1.2 hours and for 6 hours are higher than 55%.

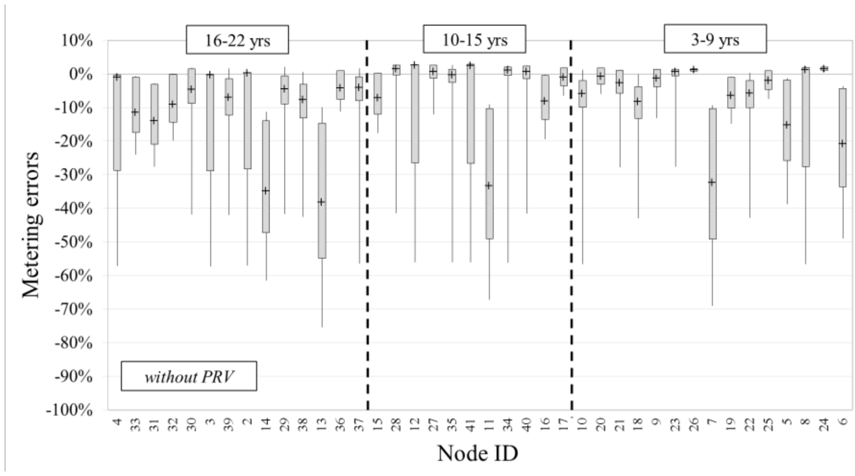

Figure A.8 Quantiles of the metering errors evaluated for each node of the network without PRV

Observing simulation results for most part of the nodes the 25th quantile of metering errors was around -30% and the 75th quantile is above zero (meaning that the metering error is in favour of the water manager). That condition is due to the high pressures in the network that permit the rapid filling of the private tanks, reducing the metering errors, and to the low flows that enter the tank after the filling, increasing the apparent losses in the rest of the day. The water level in those tanks does not fall much during the day, the valve opens only partially and the flow rate passing through the meter and entering the tank is in the range where the apparent losses are high. Water meters were divided in age groups in order to investigate the impact of aging on apparent losses: the picture provided in complex and the trend cannot easily be perceived because of many water meters providing higher apparent losses than expected; anyway, looking at the median daily losses, the age class between 15 and 22 years provide much higher apparent losses than the other groups. The age class between 10 and 15 years is the best performing (in the average) probably because of a better turbine technology installed in those years.

The same analysis was carried out making the hypothesis that a PRV is installed in the inlet node, reducing the pressure in that node from 43 to 20 m (Figure A.9). The reduction of pressure slows down the private tanks filling for some nodes. This results in several cyclical filling and empting processes of the tanks; the related float valves are completely open for a longer time and allow water to pass through the meter at flow rates higher than the starting flow. As results, the 25% quantile of metering errors is reduced in absolute values. However, with regard to the median metering errors, pressure reduction increases the metering error significantly in some nodes (for instance, nodes 14 and 33) as shown in Figure A.9 and in Figure A.10, which represents the difference between the average metering errors evaluated for each network node without and with PRV. As demonstrated in Fontanazza et al. (2010a) ageing and pressure are both relevant parameters determining meter starting flow. The first is related to starting flow by a non-linear law, with starting flow progressively increasing with the age of the meter. Network pressure also has a linear influence on starting flow, with its effect highest for newer meters and progressively masked by wear and tear during meter ageing: the meter starting flow increases as pressure reduces and this behaviour can greatly affect apparent losses due to metering under-registration.

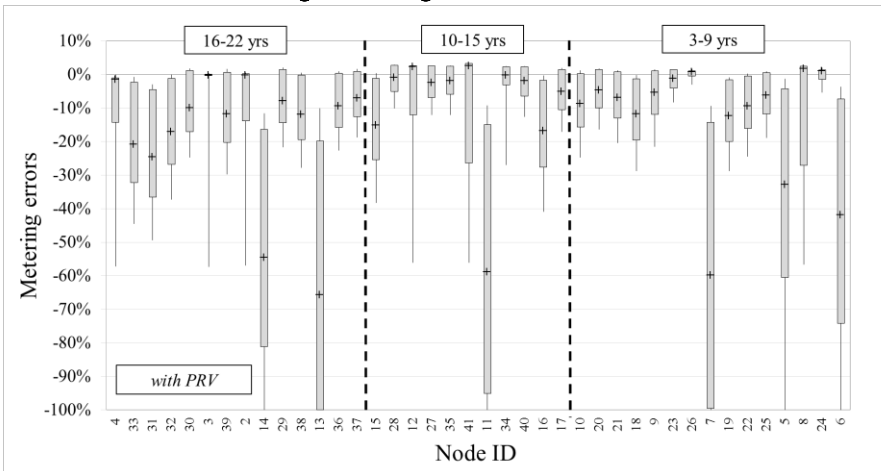

Figure A.9 Quantiles of the metering errors evaluated for each node of the network with PRV

As Figures A.8 and A.9 show, the distance between the 25% and the 75% quantiles of the metering errors evaluated for each network node with the presence of a PRV are often higher than those assessed when no PRV is installed, thus confirming that pressure control may have a negative impact on apparent losses. This is not a general rule as the case of node 3 may demonstrate but the analysis shows that the reduction of pressure variation in the network influences the meter error variability: if pressure is maintained in a range where metering errors are high this have a negative impact on apparent losses; in the opposite case, the presence of PRV may mitigate the apparent losses along with the real ones. The 95th quantile value of metering error is about -30 and -45%, without and with PRV, respectively (Figure A.8 and A.9). Highest apparent losses are often not related to the same network node thus demonstrating that the change in network pressure distribution may change the identification of most critical node in terms of apparent losses.

The experimental study reported in Fontanazza et al. (2010a) showed that the older age meters classes are characterised by lower relative decrease in starting flow and this behaviour is correctly represented by the model. Even if meter age is considered the most important parameter to carry out a meter substitution plan, older devices can show lower metering errors than newer depending on the hydraulic condition of the network. The correlation between meter age and the amount of apparent losses due to meter under-registration is not simple. The meter may under-register when the tank is usually full and the float valve opens as soon as tank water levels fall. Big consumers (usually condos) are characterised by large tanks and a consumption profile that is more distributed along the day; consequently the water level does not fall much, the valve opens only partially and the flow rate passing through the meter and entering the tank is very low taking to high volumes to be apparently lost (Figure A.10). Small residential consumers are often characterised by consumption profiles that are concentrated in the evening; for that small period, the tank water level drops, the float valve opens completely and water volumes flow into the tank at a high flow rate thus reducing apparent losses. Even old meters may produce small errors if network pressure and tank filling and emptying works together for increasing the flows passing through the meter.

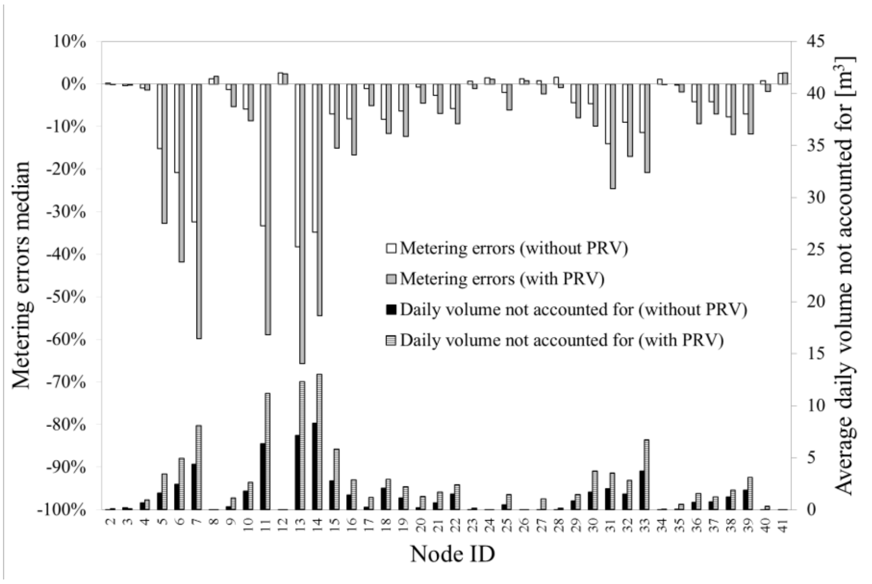

Figure A.10 Difference between the average metering error evaluated for each network node without and with PRV.

Those results revealed how the complexity of the physical phenomena associated with metering errors in ageing flow meters does not allow meter replacement to be guided by single parameters, such as the meter age or the total metered volume. As discussed in Fontanazza et al. (2012), a composite indicator taking into account three of the most influential parameters that can affect metering accuracy such as the meter age, the total metered volume, and the network pressure could be an useful performance-based tool for prioritising water meter replacement in an urban distribution network.

This study highlighted that pressure control, by means of PRVs, may have a negative side effect in the form of an increase in apparent losses. The water meter under-registration increases even up to about 50% when a PRV is installed in the inlet node of the DMA.

PRVs may act negatively on apparent losses was demonstrated in the paper but another conclusion was the fact that the stabilisation of pressure may even act positively on metering errors: if pressure is stabilised in a range where flows passing through the meter are higher, apparent losses are reduced by the presence of the PRV. This evidence is although limited to few nodes and in general the reduction of network pressures takes the increase of apparent losses as a consequence Such an effect should be considered when water loss reduction campaigns are designed by the utilities that are increasingly interested in conserving water by reducing pressure and a detailed analysis should be carried out to evaluate the local implications of pressure management. Furthermore, if the flow entering a user's water system is controlled by a tank as happens for the analysed case study, the combined influence of pressure, meter age and the tank filling process significantly affects metering errors: these can be negligible if the tank fills and empties cyclically, the water meter is new and the pressure is high; the opposite occurs if the tank does not ever completely empty, the meter is old and the pressure is low.

A.5 Energy recovery

The potential energy recovery from the use of centrifugal Pumps As Turbines (PATs) in a water distribution network characterized by the presence of private tanks was evaluated (Puleo et al. 2013). The model was applied to the Noce-Uditore DMA. Three different scenarios were analysed and compared with a baseline scenario (Scenario 0 – no PAT installed) to identify the system configuration with added PATs that permits the maximal energy recovery without penalizing the hydraulic network performance. Scenario 1 simulates the presence of PAT devices at the district inlet (node 1) only. This scenario suggests the adoption of PAT devices in place of a more traditional PRV to control pressure on distribution network. Scenario 2 simulates the presence of PAT devices in each service connection and not in the district inlet node. Figure A.11 show the condominium-user systems considering the hypothetical installation of the PAT, while Figure A.12 refers to residential-user systems.

Scenario 3 simulates an intermediate condition where PAT devices are installed on each of the five condominium service connections and on the district inlet node. Such a solution was investigated to evaluate the possible benefit of a combination of centralized and decentralized devices. In scenarios involving PAT on service connections, the specification of PAT operational parameters was also evaluated by means of Monte Carlo Analysis.

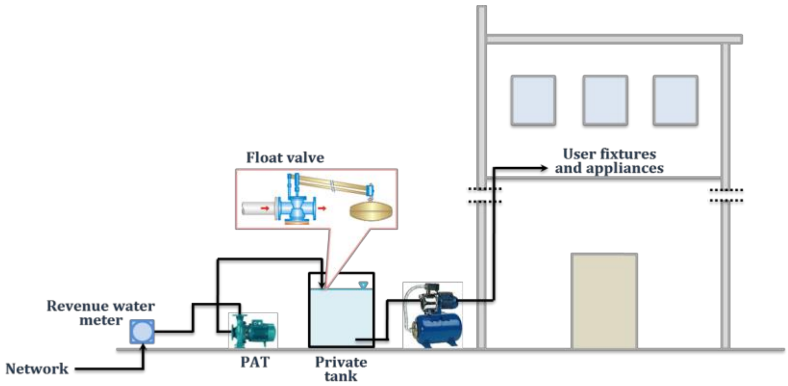

Figure A.11 A schematic of the modelled system considering the PAT installation with an underground tank

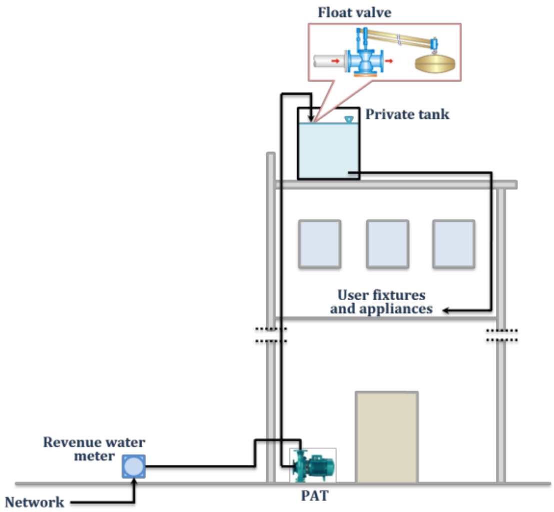

Figure A.12 A schematic of the modelled system considering the PAT installation with a roof tank

PATs were selected according to the average operating condition at the service connections and the district inlet, which were known from the network hydraulic simulation (Scenario 0 – no PAT installed). In the DMA it was possible to identify three different PAT types according to the available head and flow rates. Characteristic curves shift slightly at each time step depending on the PAT rotational speed.

For each scenario, Figure A.13 shows the obtained results in terms of stored water volume in the private tank, corresponding to three service connections taken as examples (see Figure A.2): a condominium with an underground tank (node 4) and two residential users with roof top tanks (nodes 27 and 34). Some nodes (including node 34) demonstrate a general increase or decrease of the stored volume that is due to the combination of variable pressure levels during the monitored period. Over the long term, this is not a persistent behaviour but, realistically, a long-term filling and emptying cycle.

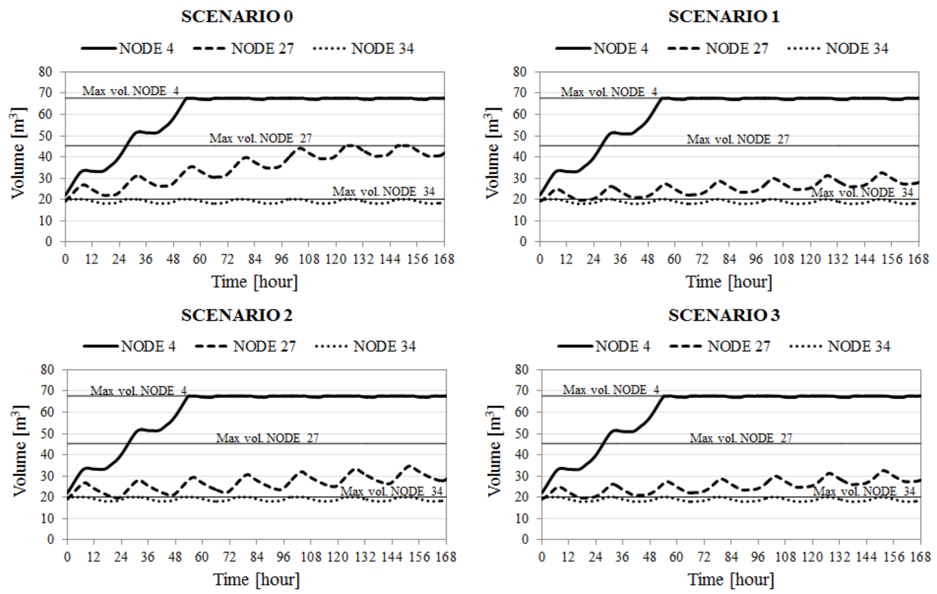

Figure A.13 Stored water volume in the private tanks linked to a condominium with an underground tank (node 4) and two residential users with roof top tanks (nodes 27 and 34).

Analysis of Figure A.13 identifies the following considerations:

– The PAT effect in terms of the stored volume is more tangible for node 27, especially when a centralized PAT is simulated at the DMA inlet node (Scenarios 1 and 3).

- The PAT does not reduce the ability of some users to fill their reservoirs, but the filling rate is lower.
- Regarding one of the condominium nodes (node 4): the volumes stored in tanks are quite similar over the scenarios because the pressure available on the tank valve is quite high and even the presence of a local PAT (Scenario 2 and 3) does not reduce flow in a significant way.
- Finally, considering the single user residential node (node 34), the differences between scenarios are not relevant due to the low flows serving the user so the presence of the PAT is shown not be affecting inflows in a significant fashion.

Figure A.14 illustrates the energy production during the day in Scenario 2 at three of the reference nodes. Node 4 is a condominium and the energy production is notably higher because of the higher flow through the service connection; energy production is at a maximum during the night because the ball valve is totally open; while, during the day, energy production is reduced due to the lower flow and pressure on the service connection. The tank is always supplied by the network and the energy production is continuous. The other two nodes are single users and the energy production is lower and discontinuous during some parts of the day because the tank is totally full and/or the network pressure is too low for supplying the tank. The usability of produced energy is thus greater for the condominiums both for the quantity of supplied power and the continuity of its production.

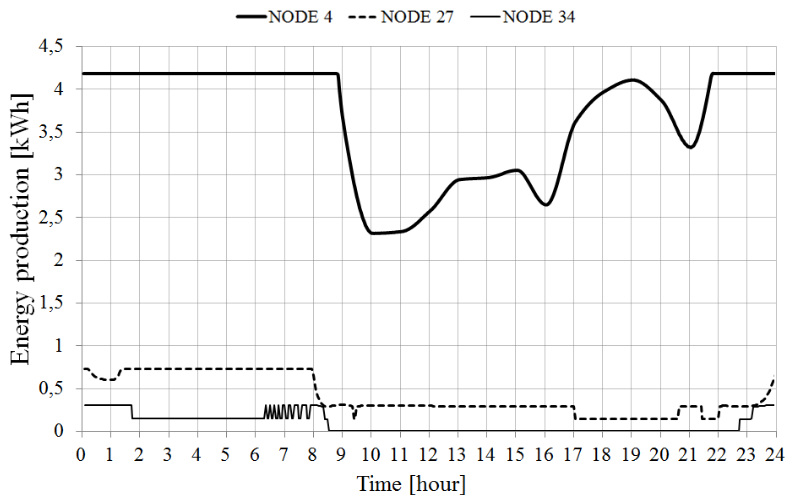

Figure A.14 Energy production from nodes 4, 27 and 34 in Scenario 2 during a 24-hour simulation.

Figures A.15 and A.16 show the results of Monte Carlo simulations for the single user service connections and for condominiums, respectively, under Scenario 2. For all 100 random simulations, energy production was computed offering a statistical interpretation of obtainable energy depending on local regulation of each PAT. The graphs are presented in terms of box and whisker plots in order to show the optimized selection of PAT regulation: the black point showing the maximum average energy production, the average condition and 25th and 75th percentiles regarding PAT regulation. Single users may produce a maximum daily energy output ranging between 1.2 kWh per day and 7.2 kWh per day, depending on the combination of flows and available pressure at the node (Figure A.15). Figure A.15 shows the results related to 27 service connections only: the other eight user connections were omitted from the graph owing to their negligible energy production – linked to the low flow rates supplied to these nodes during the day. Condominium users clearly provide higher energy productions between 12 kWh per day and 108 kWh per day (Figure A.16). The analysis of the results show that for many single users, the application of PAT may be not justified by the energy production, being too low to be economically viable, while such application may be more appropriate for condominium users that may offset part of their energy consumption.

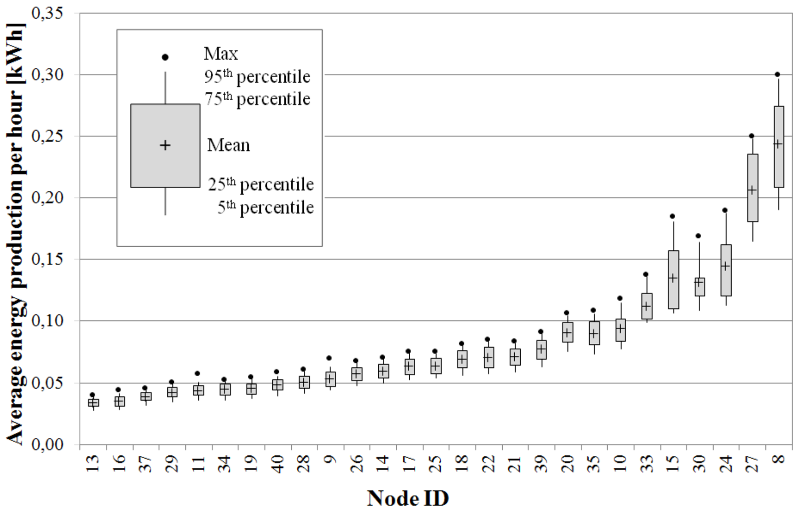

Figure A.15 Average energy production per hour for the single user service connections obtained by performing hydraulic network model simulations with random PAT regulation permutations.

Scenario 3, in which condominium PATs were combined with a centralized installation at the network inlet, resulted in a graph similar to Figure 8: net energy production was lower by about 23% depending on the presence of the centralized PAT.

Comparing the three scenarios on an annual basis:

- Scenario 1 was the least energy productive because a large part of available energy was wasted in guaranteeing sufficient pressure to those nodes characterised by low pressure;
- Scenario 2 provides a significant increase in production but the energy produced is often from PATs operating at low efficiency due to low flows supplying the user;
- Scenario 3 provides a good compromise thus giving the largest energy production on an annual basis (Table A.3). In general, the centralised PAT cut inlet pressures to a level that did not compromise user supply so the average network pressure remains higher than in Scenario 2; the surplus available energy can be then locally used by the decentralised PATs, resulting in a second cut in pressure at the service connection.

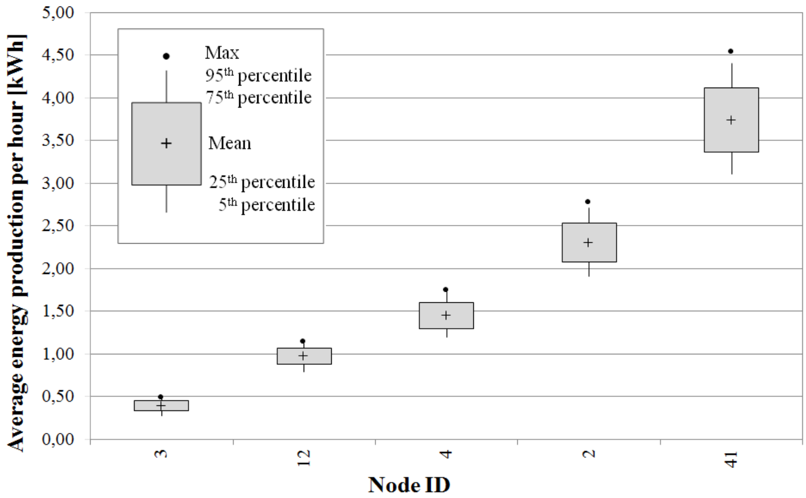

Figure A.16 Average energy production per hour for the five condominium user service connections obtained by running the hydraulic network model with random PAT regulation permutations.

Table A.3 Annual energy production in the proposed scenarios

Average annual energy production [MWh/year]	*Scenario 1*	*Scenario 2*	*Scenario 3*
DMA	60.04	97.53	139.48
For each service connection	1.88	3.05	4.36

The analysis demonstrates that the energy production on single residential service connections can be low and also discontinuous – questioning the efficacy of such energy production. Larger, condominium users demonstrate more continuous energy production owing to the fact that the underground tanks remain supplied throughout the day. The choice of a centralised solution with a PAT installed downstream of the DMA inlet node, combined with local PATs on the larger service connections, was thus shown to be the most energy efficient.

Network supply conditions and a possible increase of inlet pressure may change the efficiency of PAT systems thus changing the final choice to another scenario. The results obtained are dependent on the analysed period and on the available data so their generalisation cannot be undertaken in a rigorous fashion and further validation should be provided by additional monitoring.

APPENDIX B.1

Anytown input file

[TITLE]
ANYTOWN
example

[JUNCTIONS]

;ID	Elev	Demand	Pattern	
1	20	0	1a	;
2	20	250	1a	;
3	50	100	1a	;
4	50	100	1a	;
5	50	100	1a	;
6	80	100	1a	;
7	120	100	1a	;
8	120	100	1a	;
9	120	100	1a	;
10	50	250	1a	;
11	50	250	1a	;
12	50	250	1a	;
13	50	500	1a	;
14	50	250	1a	;
15	120	100	1a	;
16	50	250	1a	;
17	120	400	1a	;
18	120	0	1a	;
19	120	0	1a	;

[RESERVOIRS]

;ID	Head	Pattern
20	10	;

[TANKS]

;ID	Elevation	InitLevel	MinLevel	MaxLevel	Diameter	Min Vol	Vol Curve
21	215	26.5	7	35	40	0	;

[PIPES]

;ID	Node1	Node2	Length	Diameter	Roughness	Minor Loss	Status
1	1	2	100	30	130	0	Open
2	2	3	12000	12	120	0	Open
3	3	4	6000	10	120	0	Open
4	4	5	6000	10	120	0	Open
5	5	6	12000	8	120	0	Open
6	6	7	12000	8	120	0	Open
7	7	8	6000	8	120	0	Open
8	8	9	6000	8	120	0	Open
9	9	10	6000	8	120	0	Open
10	2	10	12000	12	70	0	Open
11	2	11	12000	16	70	0	Open
12	3	11	9000	12	70	0	Open
13	11	12	6000	12	70	0	Open
14	11	13	6000	10	70	0	Open
15	11	16	6000	12	70	0	Open
16	10	16	6000	8	70	0	Open
17	3	12	6000	10	120	0	Open
18	12	13	6000	10	70	0	Open
19	16	13	6000	10	70	0	Open
20	3	5	9000	10	120	0	Open
21	12	14	6000	12	70	0	Open
22	14	13	6000	10	70	0	Open
23	15	13	6000	10	70	0	Open
24	15	16	6000	12	70	0	Open
25	17	16	6000	8	120	0	Open
26	10	17	6000	10	120	0	Open
27	9	17	9000	10	130	0	Open
28	8	17	6000	10	120	0	Open
29	15	17	6000	8	120	0	Open
30	14	15	6000	10	70	0	Open
31	5	14	6000	10	120	0	Open
32	6	14	6000	10	120	0	Open
33	6	15	6000	8	120	0	Open
34	6	17	6000	8	120	0	Open
35	17	18	100	12	120	0	Open
36	18	19	100	12	120	0	Open
37	19	21	1000	12	100	0	Open

[PUMPS]

;ID	Node1	Node2	Parameter		
38	20	1	HEAD	1	;
39	20	1	HEAD	2	;
40	20	1	HEAD	3	;
41	20	1	HEAD	4	;

```
[PATTERNS]
;ID             Multipliers
1a              1.2     0.4     0.5     0.6     0.7     0.8
1a              0.9     1.0     1.1     1.2     1.2     0.4
1a              0.5     0.6     0.7     0.8     0.9     1.0
1a              1.1     1.2     1.2     0.4     0.5     0.6
;
tariff          0.024   0.024   0.024   0.024   0.024   0.024
tariff          0.024   0.12    0.12    0.12    0.12    0.12
tariff          0.12    0.12    0.12    0.12    0.12    0.12
tariff          0.12    0.12    0.12    0.12    0.12    0.12

[CURVES]
;ID PUMP        X-Value Y-Value
1               0       540
1               2500    390
1               3600    270
1               4600    140
;
2               0       560
2               2800    400
2               4000    290
2               5000    175
;
3               0       580
3               3200    415
3               4500    310
3               5750    200
;
4               0       600
4               3500    440
4               5400    335
4               6500    240
;ID EFFICIENCY          flow    efficiency
E1              0       0
E1              2250    75.6
E1              4500    94.5
E1              7200    75.6
;
E2              0       0
E2              3150    75.6
E2              5400    94.5
E2              8100    75.6
;
E3              0       0
E3              4050    75.6
E3              6300    94.5
E3              9000    75.6
;
E4              0       0
E4              4950    75.6
E4              7200    94.5
```

E4	9900	75.6	

[ENERGY]

Global Efficiency	75		
Global Price	0		
Demand Charge	0		
Pump	38	Efficiency	E1
Pump	38	Price	1
Pump	38	Pattern	tariff
Pump	39	Efficiency	E2
Pump	39	Price	1
Pump	39	Pattern	tariff
Pump	40	Efficiency	E3
Pump	40	Price	1
Pump	40	Pattern	tariff
Pump	41	Efficiency	E4
Pump	41	Price	1
Pump	41	Pattern	tariff

[TIMES]

Duration	24:00:00
Hydraulic Timestep	01:00
Quality Timestep	00:01
Pattern Timestep	01:00
Pattern Start	00:00
Report Timestep	01:00
Report Start	00:00
Start ClockTime	12:00 AM
Statistic	NONE

[REPORT]

Status	No
Summary	No
Page	0

[OPTIONS]

Units	GPM
Headloss	H-W
Specific Gravity	1
Viscosity	1
Trials	40
Accuracy	0.001
CHECKFREQ	2
MAXCHECK	10

DAMPLIMIT	0			
Unbalanced	Continue	10		
Emitter Exponent	0.5			
Quality	NONE	mg/L		
Diffusivity	1.0			
Tolerance	0.01			
[COORDINATES]				
;Node	X-Coord	Y-Coord		
1	8186.00	3379.00		
2	7682.33	3371.15		
3	7633.71	5737.44		
4	7520.26	7293.35		
5	6175.04	7568.88		
6	3321.19	6696.55		
7	1305.62	4850.97		
8	2317.45	3588.20		
9	3701.63	2892.06		
10	4846.03	3354.94		
11	6450.57	4424.64		
12	6466.77	5769.85		
13	5332.25	5332.25		
14	5094.45	6482.09		
15	4017.86	5615.96		
16	4846.03	4440.84		
17	3273.17	4464.73		
18	2422.68	4519.09		
19	2391.24	4910.54		
20	8700.60	3371.15		
21	2391.24	5230.36		
[VERTICES]				
;Link	X-Coord	Y-Coord		
38	8471.47	3551.31		
39	8432.70	3430.16		
40	8431.49	3317.50		
41	8487.22	3221.79		
[BACKDROP]				
DIMENSIONS	935.87	2613.65	9070.35	8738.70
UNITS	None			
FILE				
OFFSET	0.00	0.00		
[END]				

APPENDIX B.2

Complex network input files

In the following, the input file related to January is presented; with regard to other months, only PATTERNS are reported.

[TITLE]
Complex network (Price and Ostfeld, 2013a,b)
[JUNCTIONS]

ID	Elev	Demand	Pattern
aJ1	70	0	
aD1	70	1	aD1
aJ2	70	0	
aJ3	70	0	
aJ5	70	0	
aJ4	70	0	
aJ6	70	0	
aJ7	70	0	
aD4	70	1	aD4
aD3	70	1	aD3
aD2	70	1	aD2
cJ	100	0	
cD	100	1	cD
bJ	150	0	
bD	150	1	bD
aW1	70	0	
cP1	70	0	
bP1	70	0	
aP1	70	0	
cP2	70	0	
aP2	70	0	
aW2	70	0	
bP2	70	0	

[RESERVOIRS]

ID	Head	Pattern
aSp	50	aSp
aSw	0	aSw

[TANKS]

ID	Elevation	InitLevel	Min Level	Max Level	Diameter	Min Vol	Vol Curve
aR	90	0	0	10	50	0	aR
cR	140	0	0	10	50	0	cR
bR	190	0	0	10	50	0	bR

[PIPES]

ID	Node1	Node2	Length	Diameter	Roughness	Minor Loss	Status
1	aJ1	aR	1000	600	120	0	Open
2	aJ1	aD1	1000	400	120	0	Open
3	aJ1	aJ2	1000	600	120	0	Open
4	aJ2	aJ3	1000	400	120	0	Open
5	aJ2	aJ5	1000	400	120	0	Open
6	aJ5	aJ4	1000	400	120	0	Open
7	aJ3	aJ4	1000	400	120	0	Open
8	aJ4	aJ6	1000	400	120	0	Open
9	aJ6	aD3	1000	300	120	0	Open
10	aJ6	aJ7	1000	400	120	0	Open
11	aJ7	aJ5	1000	400	120	0	Open
12	aJ7	aD2	1000	300	120	0	Open
13	cJ	cD	1000	300	120	0	Open
14	cJ	cR	1000	300	120	0	Open
15	aJ4	aD4	1000	300	120	0	Open
16	bJ	bD	1000	450	120	0	Open
17	bJ	bR	1000	450	120	0	Open
18	aJ3	bP1	1000	400	120	0	Open
19	aSw	aW1	1000	400	120	0	Open
20	aJ7	cP1	1000	300	120	0	Open
24	aSp	aP1	1000	600	120	0	Open
26	cP2	cJ	1000	300	120	0	Open
27	aP2	aJ1	1000	600	120	0	Open
28	aW2	aJ3	1000	400	120	0	Open
29	bP2	bJ	1000	400	120	0	Open

[PUMPS]

ID	Node1	Node2	Parameters	
21	cP1	cP2	HEAD	cP
22	bP1	bP2	HEAD	bP
23	aW1	aW2	HEAD	aW
25	aP1	aP2	HEAD	aP

[PATTERNS]

ID	Multipliers					
aD1	0	0	0	0	0	0
aD1	0	163.71	163.71	163.71	163.71	163.71
aD1	163.71	163.71	163.71	163.71	163.71	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	162.58	162.58	162.58	162.58	162.58

aD1	162.58	162.58	162.58	162.58	162.58	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	155.81	155.81	155.81	155.81	155.81
aD1	155.81	155.81	155.81	155.81	155.81	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	158.06	158.06	158.06	158.06	158.06
aD1	158.06	158.06	158.06	158.06	158.06	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	168.23	168.23	168.23	168.23	168.23
aD1	168.23	168.23	168.23	168.23	168.23	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	175	175	175	175	175
aD1	175	175	175	175	175	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	145.65	145.65	145.65	145.65	145.65
aD1	145.65	145.65	145.65	145.65	145.65	0
aD1	0	0	0	0	0	0

aD2	0	0	0	0	0	0
aD2	0	98.23	98.23	98.23	98.23	98.23
aD2	98.23	98.23	98.23	98.23	98.23	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	97.55	97.55	97.55	97.55	97.55
aD2	97.55	97.55	97.55	97.55	97.55	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	93.48	93.48	93.48	93.48	93.48
aD2	93.48	93.48	93.48	93.48	93.48	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	94.84	94.84	94.84	94.84	94.84
aD2	94.84	94.84	94.84	94.84	94.84	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	100.94	100.94	100.94	100.94	100.94
aD2	100.94	100.94	100.94	100.94	100.94	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	105	105	105	105	105
aD2	105	105	105	105	105	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	87.39	87.39	87.39	87.39	87.39
aD2	87.39	87.39	87.39	87.39	87.39	0
aD2	0	0	0	0	0	0

aD3	0	0	0	0	0	0
aD3	0	130.97	130.97	130.97	130.97	130.97
aD3	130.97	130.97	130.97	130.97	130.97	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	130.06	130.06	130.06	130.06	130.06
aD3	130.06	130.06	130.06	130.06	130.06	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	124.65	124.65	124.65	124.65	124.65
aD3	124.65	124.65	124.65	124.65	124.65	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	126.45	126.45	126.45	126.45	126.45
aD3	126.45	126.45	126.45	126.45	126.45	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	134.58	134.58	134.58	134.58	134.58
aD3	134.58	134.58	134.58	134.58	134.58	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	140	140	140	140	140
aD3	140	140	140	140	140	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	116.52	116.52	116.52	116.52	116.52
aD3	116.52	116.52	116.52	116.52	116.52	0
aD3	0	0	0	0	0	0

aD4	0	0	0	0	0	0
aD4	0	98.23	98.23	98.23	98.23	98.23
aD4	98.23	98.23	98.23	98.23	98.23	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	97.55	97.55	97.55	97.55	97.55
aD4	97.55	97.55	97.55	97.55	97.55	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	93.48	93.48	93.48	93.48	93.48
aD4	93.48	93.48	93.48	93.48	93.48	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	94.84	94.84	94.84	94.84	94.84
aD4	94.84	94.84	94.84	94.84	94.84	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	100.94	100.94	100.94	100.94	100.94
aD4	100.94	100.94	100.94	100.94	100.94	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	105	105	105	105	105
aD4	105	105	105	105	105	0

aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	87.39	87.39	87.39	87.39	87.39
aD4	87.39	87.39	87.39	87.39	87.39	0
aD4	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	245.56	245.56	245.56	245.56	245.56
bD	245.56	245.56	245.56	245.56	245.56	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	243.87	243.87	243.87	243.87	243.87
bD	243.87	243.87	243.87	243.87	243.87	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	233.71	233.71	233.71	233.71	233.71
bD	233.71	233.71	233.71	233.71	233.71	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	237.1	237.1	237.1	237.1	237.1
bD	237.1	237.1	237.1	237.1	237.1	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	252.34	252.34	252.34	252.34	252.34
bD	252.34	252.34	252.34	252.34	252.34	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	262.5	262.5	262.5	262.5	262.5
bD	262.5	262.5	262.5	262.5	262.5	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	218.47	218.47	218.47	218.47	218.47
bD	218.47	218.47	218.47	218.47	218.47	0
bD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	163.71	163.71	163.71	163.71	163.71
cD	163.71	163.71	163.71	163.71	163.71	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	162.58	162.58	162.58	162.58	162.58
cD	162.58	162.58	162.58	162.58	162.58	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	155.81	155.81	155.81	155.81	155.81
cD	155.81	155.81	155.81	155.81	155.81	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	158.06	158.06	158.06	158.06	158.06
cD	158.06	158.06	158.06	158.06	158.06	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0

cD	0	168.23	168.23	168.23	168.23	168.23
cD	168.23	168.23	168.23	168.23	168.23	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	175	175	175	175	175
cD	175	175	175	175	175	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	145.65	145.65	145.65	145.65	145.65
cD	145.65	145.65	145.65	145.65	145.65	0
cD	0	0	0	0	0	0

ElectTariff	0.3051	0.3051	0.3051	0.3051	0.3051	0.3051
ElectTariff	0.5347	0.5347	0.3051	0.3051	0.3051	0.3051
ElectTariff	0.3051	0.3051	0.3051	0.3051	0.9125	0.9125
ElectTariff	0.9125	0.9125	0.9125	0.9125	0.3051	0.3051
ElectTariff	0.3051	0.3051	0.3051	0.3051	0.3051	0.3051
ElectTariff	0.5347	0.5347	0.3051	0.3051	0.3051	0.3051
ElectTariff	0.3051	0.3051	0.3051	0.3051	0.9125	0.9125
ElectTariff	0.9125	0.9125	0.9125	0.9125	0.3051	0.3051
ElectTariff	0.3051	0.3051	0.3051	0.3051	0.3051	0.3051
ElectTariff	0.5347	0.5347	0.3051	0.3051	0.3051	0.3051
ElectTariff	0.3051	0.3051	0.3051	0.3051	0.9125	0.9125
ElectTariff	0.9125	0.9125	0.9125	0.9125	0.3051	0.3051
ElectTariff	0.3051	0.3051	0.3051	0.3051	0.3051	0.3051
ElectTariff	0.5347	0.5347	0.3051	0.3051	0.3051	0.3051
ElectTariff	0.3051	0.3051	0.3051	0.3051	0.9125	0.9125
ElectTariff	0.9125	0.9125	0.9125	0.9125	0.3051	0.3051
ElectTariff	0.3051	0.3051	0.3051	0.3051	0.3051	0.3051
ElectTariff	0.5347	0.5347	0.3051	0.3051	0.3051	0.3051
ElectTariff	0.3051	0.3051	0.3051	0.3051	0.9125	0.9125
ElectTariff	0.9125	0.9125	0.9125	0.9125	0.3051	0.3051
ElectTariff	0.3051	0.3051	0.3051	0.3051	0.3051	0.3051
ElectTariff	0.3051	0.3051	0.3051	0.3051	0.3051	0.3051
ElectTariff	0.3051	0.3051	0.3051	0.3051	0.5347	0.5347
ElectTariff	0.5347	0.5347	0.3051	0.3051	0.3051	0.3051
ElectTariff	0.3051	0.3051	0.3051	0.3051	0.3051	0.3051
ElectTariff	0.3051	0.3051	0.3051	0.3051	0.3051	0.3051
ElectTariff	0.3051	0.3051	0.3051	0.3051	0.3051	0.9125
ElectTariff	0.9125	0.5347	0.5347	0.3051	0.3051	0.3051

[CURVES]

ID	X-Value	Y-Value
PUMP:		
aP	1250	47
PUMP:		
aW	1000	99
PUMP:		
bP	400	101
PUMP:		
cP	250	55

VOLUME:			
aR	0	0	
aR	10	8000	
VOLUME:			
bR	0	0	
bR	10	3000	
VOLUME:			
cR	0	0	
cR	10	2000	
EFFICIENCY:			
Eff	0	75	
Eff	1000	75	

[ENERGY]			
Global Efficiency		75	
Global	Price	0	
Demand	Charge	0	
Pump	21	Efficiency	Eff
Pump	21	Price	1
Pump	21	Pattern	Elect Tariff
Pump	22	Efficiency	Eff
Pump	22	Price	1
Pump	22	Pattern	Elect Tariff
Pump	25	Efficiency	Eff
Pump	25	Price	1
Pump	25	Pattern	Elect Tariff

[TIMES]		
Duration	168	
Hydraulic Timestep	01:00	
Quality Timestep	00:05	
Pattern Timestep	01:00	
Pattern Start	00:00	
Report Timestep	01:00	
Report Start	00:00	
Start ClockTime	12	am
Statistic	None	

[REPORT]

Status	No
Summary	No
Page	0

[OPTIONS]

Units	CMH	
Headloss	H-W	
Specific Gravity	1	
Viscosity	1	
Trials	100	
Accuracy	0.001	
CHECKFREQ	2	
MAXCHECK	10	
DAMPLIMIT	0	
Unbalanced	Continue	10
Demand Multiplier	1.0	
Emitter Exponent	0.5	
Quality	None	mg/L
Diffusivity	1	
Tolerance	0.01	

[COORDINATES]

Node	X-Coord	Y-Coord
aJ1	2064.87	7357.59
aD1	2064.87	6091.77
aJ2	3457.28	7357.59
aJ3	4691.46	7357.59
aJ5	3457.28	6044.30
aJ4	4691.46	6044.30
aJ6	4691.46	4651.90
aJ7	3457.28	4651.90
aD4	5855.93	6050.85
aD3	4691.46	3417.72
aD2	3457.28	3386.08
cJ	-308.54	4651.90
cD	-308.54	3481.01
bJ	7919.30	7357.59
bD	7919.30	6044.30
aW1	4691.46	8813.29
cP1	2159.81	4651.90
bP1	5893.99	7357.59
aP1	-403.48	7357.59
cP2	893.99	4651.90
aP2	909.81	7357.59
aW2	4691.46	8101.27
bP2	6922.47	7357.59
aSp	-1811.71	7357.59
aSw	4691.46	9604.43

aR	2064.87	8623.42
cR	-1732.59	4651.90
bR	7919.30	8670.89

[VERTICES]

Link	X-Coord	Y-Coord

[LABELS]

X-Coord	Y-Coord	Label	&	Anchor	Node

[BACKDROP]

DIMENSIONS	0.00	0.00	10000.0	10000.0
UNITS	None			
FILE				
OFFSET	0.00	0.00		

[END]

February

[PATTERNS]

ID	Multipliers					
aD1	0	0	0	0	0	0
aD1	0	181.25	181.25	181.25	181.25	181.25
aD1	181.25	181.25	181.25	181.25	181.25	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	180	180	180	180	180
aD1	180	180	180	180	180	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	172.5	172.5	172.5	172.5	172.5
aD1	172.5	172.5	172.5	172.5	172.5	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	175	175	175	175	175
aD1	175	175	175	175	175	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	186.25	186.25	186.25	186.25	186.25
aD1	186.25	186.25	186.25	186.25	186.25	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	193.75	193.75	193.75	193.75	193.75
aD1	193.75	193.75	193.75	193.75	193.75	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	161.25	161.25	161.25	161.25	161.25
aD1	161.25	161.25	161.25	161.25	161.25	0

aD1	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	108.75	108.75	108.75	108.75	108.75
aD2	108.75	108.75	108.75	108.75	108.75	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	108	108	108	108	108
aD2	108	108	108	108	108	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	103.5	103.5	103.5	103.5	103.5
aD2	103.5	103.5	103.5	103.5	103.5	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	105	105	105	105	105
aD2	105	105	105	105	105	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	111.75	111.75	111.75	111.75	111.75
aD2	111.75	111.75	111.75	111.75	111.75	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	116.25	116.25	116.25	116.25	116.25
aD2	116.25	116.25	116.25	116.25	116.25	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	96.75	96.75	96.75	96.75	96.75
aD2	96.75	96.75	96.75	96.75	96.75	0
aD2	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	145	145	145	145	145
aD3	145	145	145	145	145	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	144	144	144	144	144
aD3	144	144	144	144	144	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	138	138	138	138	138
aD3	138	138	138	138	138	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	140	140	140	140	140
aD3	140	140	140	140	140	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	149	149	149	149	149
aD3	149	149	149	149	149	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0

aD3	0	155	155	155	155	155
aD3	155	155	155	155	155	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	129	129	129	129	129
aD3	129	129	129	129	129	0
aD3	0	0	0	0	0	0

aD4	0	0	0	0	0	0
aD4	0	108.75	108.75	108.75	108.75	108.75
aD4	108.75	108.75	108.75	108.75	108.75	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	108	108	108	108	108
aD4	108	108	108	108	108	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	103.5	103.5	103.5	103.5	103.5
aD4	103.5	103.5	103.5	103.5	103.5	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	105	105	105	105	105
aD4	105	105	105	105	105	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	111.75	111.75	111.75	111.75	111.75
aD4	111.75	111.75	111.75	111.75	111.75	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	116.25	116.25	116.25	116.25	116.25
aD4	116.25	116.25	116.25	116.25	116.25	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	96.75	96.75	96.75	96.75	96.75
aD4	96.75	96.75	96.75	96.75	96.75	0
aD4	0	0	0	0	0	0

bD	0	0	0	0	0	0
bD	0	271.88	271.88	271.88	271.88	271.88
bD	271.88	271.88	271.88	271.88	271.88	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	270	270	270	270	270
bD	270	270	270	270	270	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	258.75	258.75	258.75	258.75	258.75
bD	258.75	258.75	258.75	258.75	258.75	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	262.5	262.5	262.5	262.5	262.5
bD	262.5	262.5	262.5	262.5	262.5	0

bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	279.37	279.37	279.37	279.37	279.37
bD	279.37	279.37	279.37	279.37	279.37	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	290.63	290.63	290.63	290.63	290.63
bD	290.63	290.63	290.63	290.63	290.63	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	241.87	241.87	241.87	241.87	241.87
bD	241.87	241.87	241.87	241.87	241.87	0
bD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	181.25	181.25	181.25	181.25	181.25
cD	181.25	181.25	181.25	181.25	181.25	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	180	180	180	180	180
cD	180	180	180	180	180	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	172.5	172.5	172.5	172.5	172.5
cD	172.5	172.5	172.5	172.5	172.5	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	175	175	175	175	175
cD	175	175	175	175	175	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	186.25	186.25	186.25	186.25	186.25
cD	186.25	186.25	186.25	186.25	186.25	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	193.75	193.75	193.75	193.75	193.75
cD	193.75	193.75	193.75	193.75	193.75	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	161.25	161.25	161.25	161.25	161.25
cD	161.25	161.25	161.25	161.25	161.25	0
cD	0	0	0	0	0	0
ElectTariff	0.3051	0.3051	0.3051	0.3051	0.3051	0.3051
ElectTariff	0.5347	0.5347	0.3051	0.3051	0.3051	0.3051
ElectTariff	0.3051	0.3051	0.3051	0.3051	0.9125	0.9125
ElectTariff	0.9125	0.9125	0.9125	0.9125	0.3051	0.3051
ElectTariff	0.3051	0.3051	0.3051	0.3051	0.3051	0.3051
ElectTariff	0.5347	0.5347	0.3051	0.3051	0.3051	0.3051
ElectTariff	0.3051	0.3051	0.3051	0.3051	0.9125	0.9125
ElectTariff	0.9125	0.9125	0.9125	0.9125	0.3051	0.3051
ElectTariff	0.3051	0.3051	0.3051	0.3051	0.3051	0.3051

ElectTariff	0.5347	0.5347	0.3051	0.3051	0.3051	0.3051
ElectTariff	0.3051	0.3051	0.3051	0.3051	0.9125	0.9125
ElectTariff	0.9125	0.9125	0.9125	0.9125	0.3051	0.3051
ElectTariff	0.3051	0.3051	0.3051	0.3051	0.3051	0.3051
ElectTariff	0.5347	0.5347	0.3051	0.3051	0.3051	0.3051
ElectTariff	0.3051	0.3051	0.3051	0.3051	0.9125	0.9125
ElectTariff	0.9125	0.9125	0.9125	0.9125	0.3051	0.3051
ElectTariff	0.3051	0.3051	0.3051	0.3051	0.3051	0.3051
ElectTariff	0.5347	0.5347	0.3051	0.3051	0.3051	0.3051
ElectTariff	0.3051	0.3051	0.3051	0.3051	0.9125	0.9125
ElectTariff	0.9125	0.9125	0.9125	0.9125	0.3051	0.3051
ElectTariff	0.3051	0.3051	0.3051	0.3051	0.3051	0.3051
ElectTariff	0.3051	0.3051	0.3051	0.3051	0.3051	0.3051
ElectTariff	0.3051	0.3051	0.3051	0.3051	0.5347	0.5347
ElectTariff	0.5347	0.5347	0.3051	0.3051	0.3051	0.3051
ElectTariff	0.3051	0.3051	0.3051	0.3051	0.3051	0.3051
ElectTariff	0.3051	0.3051	0.3051	0.3051	0.3051	0.3051
ElectTariff	0.3051	0.3051	0.3051	0.3051	0.3051	0.9125
ElectTariff	0.9125	0.5347	0.5347	0.3051	0.3051	0.3051

March

[PATTERNS]

ID	Multipliers					
aD1	0	0	0	0	0	0
aD1	0	196.45	196.45	196.45	196.45	196.45
aD1	196.45	196.45	196.45	196.45	196.45	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	195.1	195.1	195.1	195.1	195.1
aD1	195.1	195.1	195.1	195.1	195.1	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	186.97	186.97	186.97	186.97	186.97
aD1	186.97	186.97	186.97	186.97	186.97	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	189.68	189.68	189.68	189.68	189.68
aD1	189.68	189.68	189.68	189.68	189.68	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	201.87	201.87	201.87	201.87	201.87
aD1	201.87	201.87	201.87	201.87	201.87	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	210	210	210	210	210
aD1	210	210	210	210	210	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0

aD1	0	174.77	174.77	174.77	174.77	174.77
aD1	174.77	174.77	174.77	174.77	174.77	0
aD1	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	117.87	117.87	117.87	117.87	117.87
aD2	117.87	117.87	117.87	117.87	117.87	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	117.06	117.06	117.06	117.06	117.06
aD2	117.06	117.06	117.06	117.06	117.06	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	112.18	112.18	112.18	112.18	112.18
aD2	112.18	112.18	112.18	112.18	112.18	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	113.81	113.81	113.81	113.81	113.81
aD2	113.81	113.81	113.81	113.81	113.81	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	121.12	121.12	121.12	121.12	121.12
aD2	121.12	121.12	121.12	121.12	121.12	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	126	126	126	126	126
aD2	126	126	126	126	126	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	104.86	104.86	104.86	104.86	104.86
aD2	104.86	104.86	104.86	104.86	104.86	0
aD2	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	157.16	157.16	157.16	157.16	157.16
aD3	157.16	157.16	157.16	157.16	157.16	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	156.08	156.08	156.08	156.08	156.08
aD3	156.08	156.08	156.08	156.08	156.08	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	149.57	149.57	149.57	149.57	149.57
aD3	149.57	149.57	149.57	149.57	149.57	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	151.74	151.74	151.74	151.74	151.74
aD3	151.74	151.74	151.74	151.74	151.74	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	161.5	161.5	161.5	161.5	161.5
aD3	161.5	161.5	161.5	161.5	161.5	0

aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	168	168	168	168	168
aD3	168	168	168	168	168	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	139.82	139.82	139.82	139.82	139.82
aD3	139.82	139.82	139.82	139.82	139.82	0
aD3	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	117.87	117.87	117.87	117.87	117.87
aD4	117.87	117.87	117.87	117.87	117.87	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	117.06	117.06	117.06	117.06	117.06
aD4	117.06	117.06	117.06	117.06	117.06	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	112.18	112.18	112.18	112.18	112.18
aD4	112.18	112.18	112.18	112.18	112.18	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	113.81	113.81	113.81	113.81	113.81
aD4	113.81	113.81	113.81	113.81	113.81	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	121.12	121.12	121.12	121.12	121.12
aD4	121.12	121.12	121.12	121.12	121.12	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	126	126	126	126	126
aD4	126	126	126	126	126	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	104.86	104.86	104.86	104.86	104.86
aD4	104.86	104.86	104.86	104.86	104.86	0
aD4	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	294.68	294.68	294.68	294.68	294.68
bD	294.68	294.68	294.68	294.68	294.68	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	292.65	292.65	292.65	292.65	292.65
bD	292.65	292.65	292.65	292.65	292.65	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	280.45	280.45	280.45	280.45	280.45
bD	280.45	280.45	280.45	280.45	280.45	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0

bD	0	284.52	284.52	284.52	284.52	284.52
bD	284.52	284.52	284.52	284.52	284.52	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	302.81	302.81	302.81	302.81	302.81
bD	302.81	302.81	302.81	302.81	302.81	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	315	315	315	315	315
bD	315	315	315	315	315	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	262.16	262.16	262.16	262.16	262.16
bD	262.16	262.16	262.16	262.16	262.16	0
bD	0	0	0	0	0	0

cD	0	0	0	0	0	0
cD	0	196.45	196.45	196.45	196.45	196.45
cD	196.45	196.45	196.45	196.45	196.45	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	195.1	195.1	195.1	195.1	195.1
cD	195.1	195.1	195.1	195.1	195.1	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	186.97	186.97	186.97	186.97	186.97
cD	186.97	186.97	186.97	186.97	186.97	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	189.68	189.68	189.68	189.68	189.68
cD	189.68	189.68	189.68	189.68	189.68	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	201.87	201.87	201.87	201.87	201.87
cD	201.87	201.87	201.87	201.87	201.87	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	210	210	210	210	210
cD	210	210	210	210	210	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	174.77	174.77	174.77	174.77	174.77
cD	174.77	174.77	174.77	174.77	174.77	0
cD	0	0	0	0	0	0

ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.3407	0.3407	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194

ElectTariff	0.4194	0.4194	0.3407	0.3407	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.3407	0.3407	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.3407	0.3407	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.3407	0.3407	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.3407	0.3407	0.3407	0.3407	0.3407	0.3407
ElectTariff	0.3407	0.3407	0.3407	0.3407	0.3407	0.3407
ElectTariff	0.3407	0.3407	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.3407
ElectTariff	0.3407	0.3407	0.3407	0.2698	0.2698	0.2698

April

[PATTERNS]

ID	Multipliers					
aD1	0	0	0	0	0	0
aD1	0	270.67	270.67	270.67	270.67	270.67
aD1	270.67	270.67	270.67	270.67	270.67	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	268.8	268.8	268.8	268.8	268.8
aD1	268.8	268.8	268.8	268.8	268.8	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	257.6	257.6	257.6	257.6	257.6
aD1	257.6	257.6	257.6	257.6	257.6	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	261.33	261.33	261.33	261.33	261.33
aD1	261.33	261.33	261.33	261.33	261.33	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	278.13	278.13	278.13	278.13	278.13
aD1	278.13	278.13	278.13	278.13	278.13	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	289.33	289.33	289.33	289.33	289.33
aD1	289.33	289.33	289.33	289.33	289.33	0

aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	240.8	240.8	240.8	240.8	240.8
aD1	240.8	240.8	240.8	240.8	240.8	0
aD1	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	162.4	162.4	162.4	162.4	162.4
aD2	162.4	162.4	162.4	162.4	162.4	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	161.28	161.28	161.28	161.28	161.28
aD2	161.28	161.28	161.28	161.28	161.28	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	154.56	154.56	154.56	154.56	154.56
aD2	154.56	154.56	154.56	154.56	154.56	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	156.8	156.8	156.8	156.8	156.8
aD2	156.8	156.8	156.8	156.8	156.8	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	166.88	166.88	166.88	166.88	166.88
aD2	166.88	166.88	166.88	166.88	166.88	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	173.6	173.6	173.6	173.6	173.6
aD2	173.6	173.6	173.6	173.6	173.6	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	144.48	144.48	144.48	144.48	144.48
aD2	144.48	144.48	144.48	144.48	144.48	0
aD2	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	216.53	216.53	216.53	216.53	216.53
aD3	216.53	216.53	216.53	216.53	216.53	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	215.04	215.04	215.04	215.04	215.04
aD3	215.04	215.04	215.04	215.04	215.04	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	206.08	206.08	206.08	206.08	206.08
aD3	206.08	206.08	206.08	206.08	206.08	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	209.07	209.07	209.07	209.07	209.07
aD3	209.07	209.07	209.07	209.07	209.07	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0

aD3	0	222.51	222.51	222.51	222.51	222.51
aD3	222.51	222.51	222.51	222.51	222.51	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	231.47	231.47	231.47	231.47	231.47
aD3	231.47	231.47	231.47	231.47	231.47	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	192.64	192.64	192.64	192.64	192.64
aD3	192.64	192.64	192.64	192.64	192.64	0
aD3	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	162.4	162.4	162.4	162.4	162.4
aD4	162.4	162.4	162.4	162.4	162.4	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	161.28	161.28	161.28	161.28	161.28
aD4	161.28	161.28	161.28	161.28	161.28	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	154.56	154.56	154.56	154.56	154.56
aD4	154.56	154.56	154.56	154.56	154.56	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	156.8	156.8	156.8	156.8	156.8
aD4	156.8	156.8	156.8	156.8	156.8	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	166.88	166.88	166.88	166.88	166.88
aD4	166.88	166.88	166.88	166.88	166.88	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	173.6	173.6	173.6	173.6	173.6
aD4	173.6	173.6	173.6	173.6	173.6	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	144.48	144.48	144.48	144.48	144.48
aD4	144.48	144.48	144.48	144.48	144.48	0
aD4	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	406	406	406	406	406
bD	406	406	406	406	406	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	403.2	403.2	403.2	403.2	403.2
bD	403.2	403.2	403.2	403.2	403.2	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	386.4	386.4	386.4	386.4	386.4
bD	386.4	386.4	386.4	386.4	386.4	0

bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	392	392	392	392	392
bD	392	392	392	392	392	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	417.2	417.2	417.2	417.2	417.2
bD	417.2	417.2	417.2	417.2	417.2	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	434	434	434	434	434
bD	434	434	434	434	434	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	361.2	361.2	361.2	361.2	361.2
bD	361.2	361.2	361.2	361.2	361.2	0
bD	0	0	0	0	0	0

cD	0	0	0	0	0	0
cD	0	270.67	270.67	270.67	270.67	270.67
cD	270.67	270.67	270.67	270.67	270.67	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	268.8	268.8	268.8	268.8	268.8
cD	268.8	268.8	268.8	268.8	268.8	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	257.6	257.6	257.6	257.6	257.6
cD	257.6	257.6	257.6	257.6	257.6	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	261.33	261.33	261.33	261.33	261.33
cD	261.33	261.33	261.33	261.33	261.33	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	278.13	278.13	278.13	278.13	278.13
cD	278.13	278.13	278.13	278.13	278.13	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	289.33	289.33	289.33	289.33	289.33
cD	289.33	289.33	289.33	289.33	289.33	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	240.8	240.8	240.8	240.8	240.8
cD	240.8	240.8	240.8	240.8	240.8	0
cD	0	0	0	0	0	0

ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.3407	0.3407	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698

ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.3407	0.3407	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.3407	0.3407	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.3407	0.3407	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.3407	0.3407	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.3407	0.3407	0.3407	0.3407	0.3407	0.3407
ElectTariff	0.3407	0.3407	0.3407	0.3407	0.3407	0.3407
ElectTariff	0.3407	0.3407	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.3407
ElectTariff	0.3407	0.3407	0.3407	0.2698	0.2698	0.2698

May

[PATTERNS]

ID	Multipliers					
aD1	0	0	0	0	0	0
aD1	0	294.68	294.68	294.68	294.68	294.68
aD1	294.68	294.68	294.68	294.68	294.68	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	292.65	292.65	292.65	292.65	292.65
aD1	292.65	292.65	292.65	292.65	292.65	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	280.45	280.45	280.45	280.45	280.45
aD1	280.45	280.45	280.45	280.45	280.45	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	284.52	284.52	284.52	284.52	284.52
aD1	284.52	284.52	284.52	284.52	284.52	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	302.81	302.81	302.81	302.81	302.81
aD1	302.81	302.81	302.81	302.81	302.81	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0

aD1	0	315	315	315	315	315
aD1	315	315	315	315	315	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	262.16	262.16	262.16	262.16	262.16
aD1	262.16	262.16	262.16	262.16	262.16	0
aD1	0	0	0	0	0	0

aD2	0	0	0	0	0	0
aD2	0	176.81	176.81	176.81	176.81	176.81
aD2	176.81	176.81	176.81	176.81	176.81	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	175.59	175.59	175.59	175.59	175.59
aD2	175.59	175.59	175.59	175.59	175.59	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	168.27	168.27	168.27	168.27	168.27
aD2	168.27	168.27	168.27	168.27	168.27	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	170.71	170.71	170.71	170.71	170.71
aD2	170.71	170.71	170.71	170.71	170.71	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	181.68	181.68	181.68	181.68	181.68
aD2	181.68	181.68	181.68	181.68	181.68	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	189	189	189	189	189
aD2	189	189	189	189	189	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	157.3	157.3	157.3	157.3	157.3
aD2	157.3	157.3	157.3	157.3	157.3	0
aD2	0	0	0	0	0	0

aD3	0	0	0	0	0	0
aD3	0	235.74	235.74	235.74	235.74	235.74
aD3	235.74	235.74	235.74	235.74	235.74	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	234.12	234.12	234.12	234.12	234.12
aD3	234.12	234.12	234.12	234.12	234.12	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	224.36	224.36	224.36	224.36	224.36
aD3	224.36	224.36	224.36	224.36	224.36	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	227.61	227.61	227.61	227.61	227.61
aD3	227.61	227.61	227.61	227.61	227.61	0

aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	242.25	242.25	242.25	242.25	242.25
aD3	242.25	242.25	242.25	242.25	242.25	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	252	252	252	252	252
aD3	252	252	252	252	252	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	209.73	209.73	209.73	209.73	209.73
aD3	209.73	209.73	209.73	209.73	209.73	0
aD3	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	176.81	176.81	176.81	176.81	176.81
aD4	176.81	176.81	176.81	176.81	176.81	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	175.59	175.59	175.59	175.59	175.59
aD4	175.59	175.59	175.59	175.59	175.59	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	168.27	168.27	168.27	168.27	168.27
aD4	168.27	168.27	168.27	168.27	168.27	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	170.71	170.71	170.71	170.71	170.71
aD4	170.71	170.71	170.71	170.71	170.71	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	181.68	181.68	181.68	181.68	181.68
aD4	181.68	181.68	181.68	181.68	181.68	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	189	189	189	189	189
aD4	189	189	189	189	189	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	157.3	157.3	157.3	157.3	157.3
aD4	157.3	157.3	157.3	157.3	157.3	0
aD4	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	442.02	442.02	442.02	442.02	442.02
bD	442.02	442.02	442.02	442.02	442.02	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	438.97	438.97	438.97	438.97	438.97
bD	438.97	438.97	438.97	438.97	438.97	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0

bD	0	420.68	420.68	420.68	420.68	420.68
bD	420.68	420.68	420.68	420.68	420.68	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	426.77	426.77	426.77	426.77	426.77
bD	426.77	426.77	426.77	426.77	426.77	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	454.21	454.21	454.21	454.21	454.21
bD	454.21	454.21	454.21	454.21	454.21	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	472.5	472.5	472.5	472.5	472.5
bD	472.5	472.5	472.5	472.5	472.5	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	393.24	393.24	393.24	393.24	393.24
bD	393.24	393.24	393.24	393.24	393.24	0
bD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	294.68	294.68	294.68	294.68	294.68
cD	294.68	294.68	294.68	294.68	294.68	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	292.65	292.65	292.65	292.65	292.65
cD	292.65	292.65	292.65	292.65	292.65	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	280.45	280.45	280.45	280.45	280.45
cD	280.45	280.45	280.45	280.45	280.45	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	284.52	284.52	284.52	284.52	284.52
cD	284.52	284.52	284.52	284.52	284.52	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	302.81	302.81	302.81	302.81	302.81
cD	302.81	302.81	302.81	302.81	302.81	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	315	315	315	315	315
cD	315	315	315	315	315	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	262.16	262.16	262.16	262.16	262.16
cD	262.16	262.16	262.16	262.16	262.16	0
cD	0	0	0	0	0	0
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194

ElectTariff	0.4194	0.4194	0.3407	0.3407	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.3407	0.3407	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.3407	0.3407	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.3407	0.3407	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.3407	0.3407	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.3407	0.3407	0.3407	0.3407	0.3407	0.3407
ElectTariff	0.3407	0.3407	0.3407	0.3407	0.3407	0.3407
ElectTariff	0.3407	0.3407	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.3407
ElectTariff	0.3407	0.3407	0.3407	0.2698	0.2698	0.2698

June

[PATTERNS]

ID	Multipliers					
aD1	0	0	0	0	0	0
aD1	0	304.5	304.5	304.5	304.5	304.5
aD1	304.5	304.5	304.5	304.5	304.5	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	302.4	302.4	302.4	302.4	302.4
aD1	302.4	302.4	302.4	302.4	302.4	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	289.8	289.8	289.8	289.8	289.8
aD1	289.8	289.8	289.8	289.8	289.8	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	294	294	294	294	294
aD1	294	294	294	294	294	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	312.9	312.9	312.9	312.9	312.9
aD1	312.9	312.9	312.9	312.9	312.9	0

aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	325.5	325.5	325.5	325.5	325.5
aD1	325.5	325.5	325.5	325.5	325.5	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	270.9	270.9	270.9	270.9	270.9
aD1	270.9	270.9	270.9	270.9	270.9	0
aD1	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	182.7	182.7	182.7	182.7	182.7
aD2	182.7	182.7	182.7	182.7	182.7	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	181.44	181.44	181.44	181.44	181.44
aD2	181.44	181.44	181.44	181.44	181.44	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	173.88	173.88	173.88	173.88	173.88
aD2	173.88	173.88	173.88	173.88	173.88	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	176.4	176.4	176.4	176.4	176.4
aD2	176.4	176.4	176.4	176.4	176.4	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	187.74	187.74	187.74	187.74	187.74
aD2	187.74	187.74	187.74	187.74	187.74	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	195.3	195.3	195.3	195.3	195.3
aD2	195.3	195.3	195.3	195.3	195.3	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	162.54	162.54	162.54	162.54	162.54
aD2	162.54	162.54	162.54	162.54	162.54	0
aD2	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	243.6	243.6	243.6	243.6	243.6
aD3	243.6	243.6	243.6	243.6	243.6	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	241.92	241.92	241.92	241.92	241.92
aD3	241.92	241.92	241.92	241.92	241.92	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	231.84	231.84	231.84	231.84	231.84
aD3	231.84	231.84	231.84	231.84	231.84	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0

aD3	0	235.2	235.2	235.2	235.2	235.2
aD3	235.2	235.2	235.2	235.2	235.2	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	250.32	250.32	250.32	250.32	250.32
aD3	250.32	250.32	250.32	250.32	250.32	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	260.4	260.4	260.4	260.4	260.4
aD3	260.4	260.4	260.4	260.4	260.4	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	216.72	216.72	216.72	216.72	216.72
aD3	216.72	216.72	216.72	216.72	216.72	0
aD3	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	182.7	182.7	182.7	182.7	182.7
aD4	182.7	182.7	182.7	182.7	182.7	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	181.44	181.44	181.44	181.44	181.44
aD4	181.44	181.44	181.44	181.44	181.44	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	173.88	173.88	173.88	173.88	173.88
aD4	173.88	173.88	173.88	173.88	173.88	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	176.4	176.4	176.4	176.4	176.4
aD4	176.4	176.4	176.4	176.4	176.4	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	187.74	187.74	187.74	187.74	187.74
aD4	187.74	187.74	187.74	187.74	187.74	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	195.3	195.3	195.3	195.3	195.3
aD4	195.3	195.3	195.3	195.3	195.3	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	162.54	162.54	162.54	162.54	162.54
aD4	162.54	162.54	162.54	162.54	162.54	0
aD4	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	456.75	456.75	456.75	456.75	456.75
bD	456.75	456.75	456.75	456.75	456.75	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	453.6	453.6	453.6	453.6	453.6
bD	453.6	453.6	453.6	453.6	453.6	0

bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	434.7	434.7	434.7	434.7	434.7
bD	434.7	434.7	434.7	434.7	434.7	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	441	441	441	441	441
bD	441	441	441	441	441	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	469.35	469.35	469.35	469.35	469.35
bD	469.35	469.35	469.35	469.35	469.35	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	488.25	488.25	488.25	488.25	488.25
bD	488.25	488.25	488.25	488.25	488.25	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	406.35	406.35	406.35	406.35	406.35
bD	406.35	406.35	406.35	406.35	406.35	0
bD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	304.5	304.5	304.5	304.5	304.5
cD	304.5	304.5	304.5	304.5	304.5	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	302.4	302.4	302.4	302.4	302.4
cD	302.4	302.4	302.4	302.4	302.4	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	289.8	289.8	289.8	289.8	289.8
cD	289.8	289.8	289.8	289.8	289.8	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	294	294	294	294	294
cD	294	294	294	294	294	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	312.9	312.9	312.9	312.9	312.9
cD	312.9	312.9	312.9	312.9	312.9	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	325.5	325.5	325.5	325.5	325.5
cD	325.5	325.5	325.5	325.5	325.5	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	270.9	270.9	270.9	270.9	270.9
cD	270.9	270.9	270.9	270.9	270.9	0
cD	0	0	0	0	0	0
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698

ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.3407	0.3407	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.3407	0.3407	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.3407	0.3407	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.3407	0.3407	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.3407	0.3407	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.3407	0.3407	0.3407	0.3407	0.3407	0.3407
ElectTariff	0.3407	0.3407	0.3407	0.3407	0.3407	0.3407
ElectTariff	0.3407	0.3407	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.3407
ElectTariff	0.3407	0.3407	0.3407	0.2698	0.2698	0.2698

July

[PATTERNS]

ID	Multipliers					
aD1	0	0	0	0	0	0
aD1	0	360.16	360.16	360.16	360.16	360.16
aD1	360.16	360.16	360.16	360.16	360.16	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	357.68	357.68	357.68	357.68	357.68
aD1	357.68	357.68	357.68	357.68	357.68	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	342.77	342.77	342.77	342.77	342.77
aD1	342.77	342.77	342.77	342.77	342.77	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	347.74	347.74	347.74	347.74	347.74
aD1	347.74	347.74	347.74	347.74	347.74	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0

aD1	0	370.1	370.1	370.1	370.1	370.1
aD1	370.1	370.1	370.1	370.1	370.1	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	385	385	385	385	385
aD1	385	385	385	385	385	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	320.42	320.42	320.42	320.42	320.42
aD1	320.42	320.42	320.42	320.42	320.42	0
aD1	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	216.1	216.1	216.1	216.1	216.1
aD2	216.1	216.1	216.1	216.1	216.1	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	214.61	214.61	214.61	214.61	214.61
aD2	214.61	214.61	214.61	214.61	214.61	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	205.66	205.66	205.66	205.66	205.66
aD2	205.66	205.66	205.66	205.66	205.66	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	208.65	208.65	208.65	208.65	208.65
aD2	208.65	208.65	208.65	208.65	208.65	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	222.06	222.06	222.06	222.06	222.06
aD2	222.06	222.06	222.06	222.06	222.06	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	231	231	231	231	231
aD2	231	231	231	231	231	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	192.25	192.25	192.25	192.25	192.25
aD2	192.25	192.25	192.25	192.25	192.25	0
aD2	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	288.13	288.13	288.13	288.13	288.13
aD3	288.13	288.13	288.13	288.13	288.13	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	286.14	286.14	286.14	286.14	286.14
aD3	286.14	286.14	286.14	286.14	286.14	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	274.22	274.22	274.22	274.22	274.22
aD3	274.22	274.22	274.22	274.22	274.22	0

aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	278.19	278.19	278.19	278.19	278.19
aD3	278.19	278.19	278.19	278.19	278.19	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	296.08	296.08	296.08	296.08	296.08
aD3	296.08	296.08	296.08	296.08	296.08	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	308	308	308	308	308
aD3	308	308	308	308	308	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	256.34	256.34	256.34	256.34	256.34
aD3	256.34	256.34	256.34	256.34	256.34	0
aD3	0	0	0	0	0	0

aD4	0	0	0	0	0	0
aD4	0	216.1	216.1	216.1	216.1	216.1
aD4	216.1	216.1	216.1	216.1	216.1	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	214.61	214.61	214.61	214.61	214.61
aD4	214.61	214.61	214.61	214.61	214.61	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	205.66	205.66	205.66	205.66	205.66
aD4	205.66	205.66	205.66	205.66	205.66	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	208.65	208.65	208.65	208.65	208.65
aD4	208.65	208.65	208.65	208.65	208.65	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	222.06	222.06	222.06	222.06	222.06
aD4	222.06	222.06	222.06	222.06	222.06	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	231	231	231	231	231
aD4	231	231	231	231	231	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	192.25	192.25	192.25	192.25	192.25
aD4	192.25	192.25	192.25	192.25	192.25	0
aD4	0	0	0	0	0	0

bD	0	0	0	0	0	0
bD	0	540.24	540.24	540.24	540.24	540.24
bD	540.24	540.24	540.24	540.24	540.24	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0

bD	0	536.52	536.52	536.52	536.52	536.52
bD	536.52	536.52	536.52	536.52	536.52	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	514.16	514.16	514.16	514.16	514.16
bD	514.16	514.16	514.16	514.16	514.16	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	521.61	521.61	521.61	521.61	521.61
bD	521.61	521.61	521.61	521.61	521.61	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	555.15	555.15	555.15	555.15	555.15
bD	555.15	555.15	555.15	555.15	555.15	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	577.5	577.5	577.5	577.5	577.5
bD	577.5	577.5	577.5	577.5	577.5	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	480.63	480.63	480.63	480.63	480.63
bD	480.63	480.63	480.63	480.63	480.63	0
bD	0	0	0	0	0	0

cD	0	0	0	0	0	0
cD	0	360.16	360.16	360.16	360.16	360.16
cD	360.16	360.16	360.16	360.16	360.16	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	357.68	357.68	357.68	357.68	357.68
cD	357.68	357.68	357.68	357.68	357.68	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	342.77	342.77	342.77	342.77	342.77
cD	342.77	342.77	342.77	342.77	342.77	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	347.74	347.74	347.74	347.74	347.74
cD	347.74	347.74	347.74	347.74	347.74	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	370.1	370.1	370.1	370.1	370.1
cD	370.1	370.1	370.1	370.1	370.1	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	385	385	385	385	385
cD	385	385	385	385	385	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	320.42	320.42	320.42	320.42	320.42
cD	320.42	320.42	320.42	320.42	320.42	0
cD	0	0	0	0	0	0

ElectTariff	0.2789	0.2789	0.2789	0.2789	0.2789	0.2789
ElectTariff	0.2789	0.4316	0.4316	0.4316	10.089	10.089
ElectTariff	10.089	10.089	10.089	10.089	10.089	0.4316
ElectTariff	0.4316	0.4316	0.4316	0.2789	0.2789	0.2789
ElectTariff	0.2789	0.2789	0.2789	0.2789	0.2789	0.2789
ElectTariff	0.2789	0.4316	0.4316	0.4316	10.089	10.089
ElectTariff	10.089	10.089	10.089	10.089	10.089	0.4316
ElectTariff	0.4316	0.4316	0.4316	0.2789	0.2789	0.2789
ElectTariff	0.2789	0.2789	0.2789	0.2789	0.2789	0.2789
ElectTariff	0.2789	0.4316	0.4316	0.4316	10.089	10.089
ElectTariff	10.089	10.089	10.089	10.089	10.089	0.4316
ElectTariff	0.4316	0.4316	0.4316	0.2789	0.2789	0.2789
ElectTariff	0.2789	0.2789	0.2789	0.2789	0.2789	0.2789
ElectTariff	0.2789	0.4316	0.4316	0.4316	10.089	10.089
ElectTariff	10.089	10.089	10.089	10.089	10.089	0.4316
ElectTariff	0.4316	0.4316	0.4316	0.2789	0.2789	0.2789
ElectTariff	0.2789	0.2789	0.2789	0.2789	0.2789	0.2789
ElectTariff	0.2789	0.4316	0.4316	0.4316	10.089	10.089
ElectTariff	10.089	10.089	10.089	10.089	10.089	0.4316
ElectTariff	0.4316	0.4316	0.4316	0.2789	0.2789	0.2789
ElectTariff	0.2789	0.2789	0.2789	0.2789	0.2789	0.2789
ElectTariff	0.2789	0.2789	0.2789	0.2789	0.2789	0.2789
ElectTariff	0.2789	0.2789	0.2789	0.2789	0.2789	0.2789
ElectTariff	0.2789	0.2789	0.2789	0.2789	0.2789	0.2789
ElectTariff	0.2789	0.2789	0.2789	0.2789	0.2789	0.2789
ElectTariff	0.2789	0.2789	0.2789	0.2789	0.2789	0.2789
ElectTariff	0.2789	0.2789	0.2789	0.2789	0.2789	0.2789
ElectTariff	0.2789	0.2789	0.2789	0.2789	0.2789	0.2789

August

[PATTERNS]

ID	Multipliers					
aD1	0	0	0	0	0	0
aD1	0	392.9	392.9	392.9	392.9	392.9
aD1	392.9	392.9	392.9	392.9	392.9	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	390.19	390.19	390.19	390.19	390.19
aD1	390.19	390.19	390.19	390.19	390.19	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	373.94	373.94	373.94	373.94	373.94
aD1	373.94	373.94	373.94	373.94	373.94	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	379.35	379.35	379.35	379.35	379.35
aD1	379.35	379.35	379.35	379.35	379.35	0

aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	403.74	403.74	403.74	403.74	403.74
aD1	403.74	403.74	403.74	403.74	403.74	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	420	420	420	420	420
aD1	420	420	420	420	420	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	349.55	349.55	349.55	349.55	349.55
aD1	349.55	349.55	349.55	349.55	349.55	0
aD1	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	235.74	235.74	235.74	235.74	235.74
aD2	235.74	235.74	235.74	235.74	235.74	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	234.12	234.12	234.12	234.12	234.12
aD2	234.12	234.12	234.12	234.12	234.12	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	224.36	224.36	224.36	224.36	224.36
aD2	224.36	224.36	224.36	224.36	224.36	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	227.61	227.61	227.61	227.61	227.61
aD2	227.61	227.61	227.61	227.61	227.61	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	242.25	242.25	242.25	242.25	242.25
aD2	242.25	242.25	242.25	242.25	242.25	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	252	252	252	252	252
aD2	252	252	252	252	252	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	209.73	209.73	209.73	209.73	209.73
aD2	209.73	209.73	209.73	209.73	209.73	0
aD2	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	314.32	314.32	314.32	314.32	314.32
aD3	314.32	314.32	314.32	314.32	314.32	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	312.15	312.15	312.15	312.15	312.15
aD3	312.15	312.15	312.15	312.15	312.15	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0

aD3	0	299.15	299.15	299.15	299.15	299.15
aD3	299.15	299.15	299.15	299.15	299.15	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	303.48	303.48	303.48	303.48	303.48
aD3	303.48	303.48	303.48	303.48	303.48	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	322.99	322.99	322.99	322.99	322.99
aD3	322.99	322.99	322.99	322.99	322.99	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	336	336	336	336	336
aD3	336	336	336	336	336	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	279.64	279.64	279.64	279.64	279.64
aD3	279.64	279.64	279.64	279.64	279.64	0
aD3	0	0	0	0	0	0

aD4	0	0	0	0	0	0
aD4	0	235.74	235.74	235.74	235.74	235.74
aD4	235.74	235.74	235.74	235.74	235.74	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	234.12	234.12	234.12	234.12	234.12
aD4	234.12	234.12	234.12	234.12	234.12	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	224.36	224.36	224.36	224.36	224.36
aD4	224.36	224.36	224.36	224.36	224.36	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	227.61	227.61	227.61	227.61	227.61
aD4	227.61	227.61	227.61	227.61	227.61	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	242.25	242.25	242.25	242.25	242.25
aD4	242.25	242.25	242.25	242.25	242.25	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	252	252	252	252	252
aD4	252	252	252	252	252	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	209.73	209.73	209.73	209.73	209.73
aD4	209.73	209.73	209.73	209.73	209.73	0
aD4	0	0	0	0	0	0

bD	0	0	0	0	0	0
bD	0	589.35	589.35	589.35	589.35	589.35
bD	589.35	589.35	589.35	589.35	589.35	0

bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	585.29	585.29	585.29	585.29	585.29
bD	585.29	585.29	585.29	585.29	585.29	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	560.9	560.9	560.9	560.9	560.9
bD	560.9	560.9	560.9	560.9	560.9	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	569.03	569.03	569.03	569.03	569.03
bD	569.03	569.03	569.03	569.03	569.03	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	605.61	605.61	605.61	605.61	605.61
bD	605.61	605.61	605.61	605.61	605.61	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	630	630	630	630	630
bD	630	630	630	630	630	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	524.32	524.32	524.32	524.32	524.32
bD	524.32	524.32	524.32	524.32	524.32	0
bD	0	0	0	0	0	0

cD	0	0	0	0	0	0
cD	0	392.9	392.9	392.9	392.9	392.9
cD	392.9	392.9	392.9	392.9	392.9	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	390.19	390.19	390.19	390.19	390.19
cD	390.19	390.19	390.19	390.19	390.19	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	373.94	373.94	373.94	373.94	373.94
cD	373.94	373.94	373.94	373.94	373.94	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	379.35	379.35	379.35	379.35	379.35
cD	379.35	379.35	379.35	379.35	379.35	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	403.74	403.74	403.74	403.74	403.74
cD	403.74	403.74	403.74	403.74	403.74	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	420	420	420	420	420
cD	420	420	420	420	420	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	349.55	349.55	349.55	349.55	349.55

cD	349.55	349.55	349.55	349.55	349.55	0
cD	0	0	0	0	0	0

ElectTariff	0.2789	0.2789	0.2789	0.2789	0.2789	0.2789
ElectTariff	0.2789	0.4316	0.4316	0.4316	10.089	10.089
ElectTariff	10.089	10.089	10.089	10.089	10.089	0.4316
ElectTariff	0.4316	0.4316	0.4316	0.2789	0.2789	0.2789
ElectTariff	0.2789	0.2789	0.2789	0.2789	0.2789	0.2789
ElectTariff	0.2789	0.4316	0.4316	0.4316	10.089	10.089
ElectTariff	10.089	10.089	10.089	10.089	10.089	0.4316
ElectTariff	0.4316	0.4316	0.4316	0.2789	0.2789	0.2789
ElectTariff	0.2789	0.2789	0.2789	0.2789	0.2789	0.2789
ElectTariff	0.2789	0.4316	0.4316	0.4316	10.089	10.089
ElectTariff	10.089	10.089	10.089	10.089	10.089	0.4316
ElectTariff	0.4316	0.4316	0.4316	0.2789	0.2789	0.2789
ElectTariff	0.2789	0.2789	0.2789	0.2789	0.2789	0.2789
ElectTariff	0.2789	0.4316	0.4316	0.4316	10.089	10.089
ElectTariff	10.089	10.089	10.089	10.089	10.089	0.4316
ElectTariff	0.4316	0.4316	0.4316	0.2789	0.2789	0.2789
ElectTariff	0.2789	0.2789	0.2789	0.2789	0.2789	0.2789
ElectTariff	0.2789	0.4316	0.4316	0.4316	10.089	10.089
ElectTariff	10.089	10.089	10.089	10.089	10.089	0.4316
ElectTariff	0.4316	0.4316	0.4316	0.2789	0.2789	0.2789
ElectTariff	0.2789	0.2789	0.2789	0.2789	0.2789	0.2789
ElectTariff	0.2789	0.2789	0.2789	0.2789	0.2789	0.2789
ElectTariff	0.2789	0.2789	0.2789	0.2789	0.2789	0.2789
ElectTariff	0.2789	0.2789	0.2789	0.2789	0.2789	0.2789
ElectTariff	0.2789	0.2789	0.2789	0.2789	0.2789	0.2789
ElectTariff	0.2789	0.2789	0.2789	0.2789	0.2789	0.2789
ElectTariff	0.2789	0.2789	0.2789	0.2789	0.2789	0.2789
ElectTariff	0.2789	0.2789	0.2789	0.2789	0.2789	0.2789

September

[PATTERNS]

ID	Multipliers					
aD1	0	0	0	0	0	0
aD1	0	372.17	372.17	372.17	372.17	372.17
aD1	372.17	372.17	372.17	372.17	372.17	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	369.6	369.6	369.6	369.6	369.6
aD1	369.6	369.6	369.6	369.6	369.6	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	354.2	354.2	354.2	354.2	354.2
aD1	354.2	354.2	354.2	354.2	354.2	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0

aD1	0	359.33	359.33	359.33	359.33	359.33
aD1	359.33	359.33	359.33	359.33	359.33	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	382.43	382.43	382.43	382.43	382.43
aD1	382.43	382.43	382.43	382.43	382.43	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	397.83	397.83	397.83	397.83	397.83
aD1	397.83	397.83	397.83	397.83	397.83	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	331.1	331.1	331.1	331.1	331.1
aD1	331.1	331.1	331.1	331.1	331.1	0
aD1	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	223.3	223.3	223.3	223.3	223.3
aD2	223.3	223.3	223.3	223.3	223.3	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	221.76	221.76	221.76	221.76	221.76
aD2	221.76	221.76	221.76	221.76	221.76	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	212.52	212.52	212.52	212.52	212.52
aD2	212.52	212.52	212.52	212.52	212.52	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	215.6	215.6	215.6	215.6	215.6
aD2	215.6	215.6	215.6	215.6	215.6	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	229.46	229.46	229.46	229.46	229.46
aD2	229.46	229.46	229.46	229.46	229.46	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	238.7	238.7	238.7	238.7	238.7
aD2	238.7	238.7	238.7	238.7	238.7	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	198.66	198.66	198.66	198.66	198.66
aD2	198.66	198.66	198.66	198.66	198.66	0
aD2	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	297.73	297.73	297.73	297.73	297.73
aD3	297.73	297.73	297.73	297.73	297.73	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	295.68	295.68	295.68	295.68	295.68
aD3	295.68	295.68	295.68	295.68	295.68	0

aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	283.36	283.36	283.36	283.36	283.36
aD3	283.36	283.36	283.36	283.36	283.36	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	287.47	287.47	287.47	287.47	287.47
aD3	287.47	287.47	287.47	287.47	287.47	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	305.95	305.95	305.95	305.95	305.95
aD3	305.95	305.95	305.95	305.95	305.95	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	318.27	318.27	318.27	318.27	318.27
aD3	318.27	318.27	318.27	318.27	318.27	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	264.88	264.88	264.88	264.88	264.88
aD3	264.88	264.88	264.88	264.88	264.88	0
aD3	0	0	0	0	0	0

aD4	0	0	0	0	0	0
aD4	0	223.3	223.3	223.3	223.3	223.3
aD4	223.3	223.3	223.3	223.3	223.3	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	221.76	221.76	221.76	221.76	221.76
aD4	221.76	221.76	221.76	221.76	221.76	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	212.52	212.52	212.52	212.52	212.52
aD4	212.52	212.52	212.52	212.52	212.52	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	215.6	215.6	215.6	215.6	215.6
aD4	215.6	215.6	215.6	215.6	215.6	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	229.46	229.46	229.46	229.46	229.46
aD4	229.46	229.46	229.46	229.46	229.46	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	238.7	238.7	238.7	238.7	238.7
aD4	238.7	238.7	238.7	238.7	238.7	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	198.66	198.66	198.66	198.66	198.66
aD4	198.66	198.66	198.66	198.66	198.66	0
aD4	0	0	0	0	0	0

bD	0	0	0	0	0	0

bD	0	558.25	558.25	558.25	558.25	558.25
bD	558.25	558.25	558.25	558.25	558.25	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	554.4	554.4	554.4	554.4	554.4
bD	554.4	554.4	554.4	554.4	554.4	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	531.3	531.3	531.3	531.3	531.3
bD	531.3	531.3	531.3	531.3	531.3	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	539	539	539	539	539
bD	539	539	539	539	539	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	573.65	573.65	573.65	573.65	573.65
bD	573.65	573.65	573.65	573.65	573.65	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	596.75	596.75	596.75	596.75	596.75
bD	596.75	596.75	596.75	596.75	596.75	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	496.65	496.65	496.65	496.65	496.65
bD	496.65	496.65	496.65	496.65	496.65	0
bD	0	0	0	0	0	0

cD	0	0	0	0	0	0
cD	0	372.17	372.17	372.17	372.17	372.17
cD	372.17	372.17	372.17	372.17	372.17	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	369.6	369.6	369.6	369.6	369.6
cD	369.6	369.6	369.6	369.6	369.6	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	354.2	354.2	354.2	354.2	354.2
cD	354.2	354.2	354.2	354.2	354.2	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	359.33	359.33	359.33	359.33	359.33
cD	359.33	359.33	359.33	359.33	359.33	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	382.43	382.43	382.43	382.43	382.43
cD	382.43	382.43	382.43	382.43	382.43	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	397.83	397.83	397.83	397.83	397.83
cD	397.83	397.83	397.83	397.83	397.83	0
cD	0	0	0	0	0	0

cD	0	0	0	0	0	0
cD	0	331.1	331.1	331.1	331.1	331.1
cD	331.1	331.1	331.1	331.1	331.1	0
cD	0	0	0	0	0	0

ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.3407	0.3407	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.3407	0.3407	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.3407	0.3407	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.3407	0.3407	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.3407	0.3407	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.3407	0.3407	0.3407	0.3407	0.3407	0.3407
ElectTariff	0.3407	0.3407	0.3407	0.3407	0.3407	0.3407
ElectTariff	0.3407	0.3407	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.3407
ElectTariff	0.3407	0.3407	0.3407	0.2698	0.2698	0.2698

October

[PATTERNS]

ID	Multipliers					
aD1	0	0	0	0	0	0
aD1	0	294.68	294.68	294.68	294.68	294.68
aD1	294.68	294.68	294.68	294.68	294.68	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	292.65	292.65	292.65	292.65	292.65
aD1	292.65	292.65	292.65	292.65	292.65	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	280.45	280.45	280.45	280.45	280.45
aD1	280.45	280.45	280.45	280.45	280.45	0

aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	284.52	284.52	284.52	284.52	284.52
aD1	284.52	284.52	284.52	284.52	284.52	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	302.81	302.81	302.81	302.81	302.81
aD1	302.81	302.81	302.81	302.81	302.81	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	315	315	315	315	315
aD1	315	315	315	315	315	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	262.16	262.16	262.16	262.16	262.16
aD1	262.16	262.16	262.16	262.16	262.16	0
aD1	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	176.81	176.81	176.81	176.81	176.81
aD2	176.81	176.81	176.81	176.81	176.81	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	175.59	175.59	175.59	175.59	175.59
aD2	175.59	175.59	175.59	175.59	175.59	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	168.27	168.27	168.27	168.27	168.27
aD2	168.27	168.27	168.27	168.27	168.27	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	170.71	170.71	170.71	170.71	170.71
aD2	170.71	170.71	170.71	170.71	170.71	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	181.68	181.68	181.68	181.68	181.68
aD2	181.68	181.68	181.68	181.68	181.68	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	189	189	189	189	189
aD2	189	189	189	189	189	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	157.3	157.3	157.3	157.3	157.3
aD2	157.3	157.3	157.3	157.3	157.3	0
aD2	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	235.74	235.74	235.74	235.74	235.74
aD3	235.74	235.74	235.74	235.74	235.74	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0

aD3	0	234.12	234.12	234.12	234.12	234.12
aD3	234.12	234.12	234.12	234.12	234.12	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	224.36	224.36	224.36	224.36	224.36
aD3	224.36	224.36	224.36	224.36	224.36	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	227.61	227.61	227.61	227.61	227.61
aD3	227.61	227.61	227.61	227.61	227.61	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	242.25	242.25	242.25	242.25	242.25
aD3	242.25	242.25	242.25	242.25	242.25	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	252	252	252	252	252
aD3	252	252	252	252	252	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	209.73	209.73	209.73	209.73	209.73
aD3	209.73	209.73	209.73	209.73	209.73	0
aD3	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	176.81	176.81	176.81	176.81	176.81
aD4	176.81	176.81	176.81	176.81	176.81	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	175.59	175.59	175.59	175.59	175.59
aD4	175.59	175.59	175.59	175.59	175.59	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	168.27	168.27	168.27	168.27	168.27
aD4	168.27	168.27	168.27	168.27	168.27	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	170.71	170.71	170.71	170.71	170.71
aD4	170.71	170.71	170.71	170.71	170.71	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	181.68	181.68	181.68	181.68	181.68
aD4	181.68	181.68	181.68	181.68	181.68	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	189	189	189	189	189
aD4	189	189	189	189	189	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	157.3	157.3	157.3	157.3	157.3
aD4	157.3	157.3	157.3	157.3	157.3	0
aD4	0	0	0	0	0	0

bD	0	0	0	0	0	0
bD	0	442.02	442.02	442.02	442.02	442.02
bD	442.02	442.02	442.02	442.02	442.02	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	438.97	438.97	438.97	438.97	438.97
bD	438.97	438.97	438.97	438.97	438.97	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	420.68	420.68	420.68	420.68	420.68
bD	420.68	420.68	420.68	420.68	420.68	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	426.77	426.77	426.77	426.77	426.77
bD	426.77	426.77	426.77	426.77	426.77	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	454.21	454.21	454.21	454.21	454.21
bD	454.21	454.21	454.21	454.21	454.21	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	472.5	472.5	472.5	472.5	472.5
bD	472.5	472.5	472.5	472.5	472.5	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	393.24	393.24	393.24	393.24	393.24
bD	393.24	393.24	393.24	393.24	393.24	0
bD	0	0	0	0	0	0

cD	0	0	0	0	0	0
cD	0	294.68	294.68	294.68	294.68	294.68
cD	294.68	294.68	294.68	294.68	294.68	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	292.65	292.65	292.65	292.65	292.65
cD	292.65	292.65	292.65	292.65	292.65	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	280.45	280.45	280.45	280.45	280.45
cD	280.45	280.45	280.45	280.45	280.45	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	284.52	284.52	284.52	284.52	284.52
cD	284.52	284.52	284.52	284.52	284.52	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	302.81	302.81	302.81	302.81	302.81
cD	302.81	302.81	302.81	302.81	302.81	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	315	315	315	315	315

cD	315	315	315	315	315	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	262.16	262.16	262.16	262.16	262.16
cD	262.16	262.16	262.16	262.16	262.16	0
cD	0	0	0	0	0	0

ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.3407	0.3407	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.3407	0.3407	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.3407	0.3407	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.3407	0.3407	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.3407	0.3407	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.3407	0.3407	0.3407	0.3407	0.3407	0.3407
ElectTariff	0.3407	0.3407	0.3407	0.3407	0.3407	0.3407
ElectTariff	0.3407	0.3407	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.3407
ElectTariff	0.3407	0.3407	0.3407	0.2698	0.2698	0.2698

November

[PATTERNS]

ID	Multipliers					
aD1	0	0	0	0	0	0
aD1	0	270.67	270.67	270.67	270.67	270.67
aD1	270.67	270.67	270.67	270.67	270.67	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	268.8	268.8	268.8	268.8	268.8
aD1	268.8	268.8	268.8	268.8	268.8	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0

aD1	0	257.6	257.6	257.6	257.6	257.6
aD1	257.6	257.6	257.6	257.6	257.6	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	261.33	261.33	261.33	261.33	261.33
aD1	261.33	261.33	261.33	261.33	261.33	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	278.13	278.13	278.13	278.13	278.13
aD1	278.13	278.13	278.13	278.13	278.13	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	289.33	289.33	289.33	289.33	289.33
aD1	289.33	289.33	289.33	289.33	289.33	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	240.8	240.8	240.8	240.8	240.8
aD1	240.8	240.8	240.8	240.8	240.8	0
aD1	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	162.4	162.4	162.4	162.4	162.4
aD2	162.4	162.4	162.4	162.4	162.4	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	161.28	161.28	161.28	161.28	161.28
aD2	161.28	161.28	161.28	161.28	161.28	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	154.56	154.56	154.56	154.56	154.56
aD2	154.56	154.56	154.56	154.56	154.56	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	156.8	156.8	156.8	156.8	156.8
aD2	156.8	156.8	156.8	156.8	156.8	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	166.88	166.88	166.88	166.88	166.88
aD2	166.88	166.88	166.88	166.88	166.88	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	173.6	173.6	173.6	173.6	173.6
aD2	173.6	173.6	173.6	173.6	173.6	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	144.48	144.48	144.48	144.48	144.48
aD2	144.48	144.48	144.48	144.48	144.48	0
aD2	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	216.53	216.53	216.53	216.53	216.53
aD3	216.53	216.53	216.53	216.53	216.53	0

aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	215.04	215.04	215.04	215.04	215.04
aD3	215.04	215.04	215.04	215.04	215.04	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	206.08	206.08	206.08	206.08	206.08
aD3	206.08	206.08	206.08	206.08	206.08	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	209.07	209.07	209.07	209.07	209.07
aD3	209.07	209.07	209.07	209.07	209.07	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	222.51	222.51	222.51	222.51	222.51
aD3	222.51	222.51	222.51	222.51	222.51	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	231.47	231.47	231.47	231.47	231.47
aD3	231.47	231.47	231.47	231.47	231.47	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	192.64	192.64	192.64	192.64	192.64
aD3	192.64	192.64	192.64	192.64	192.64	0
aD3	0	0	0	0	0	0

aD4	0	0	0	0	0	0
aD4	0	162.4	162.4	162.4	162.4	162.4
aD4	162.4	162.4	162.4	162.4	162.4	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	161.28	161.28	161.28	161.28	161.28
aD4	161.28	161.28	161.28	161.28	161.28	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	154.56	154.56	154.56	154.56	154.56
aD4	154.56	154.56	154.56	154.56	154.56	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	156.8	156.8	156.8	156.8	156.8
aD4	156.8	156.8	156.8	156.8	156.8	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	166.88	166.88	166.88	166.88	166.88
aD4	166.88	166.88	166.88	166.88	166.88	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	173.6	173.6	173.6	173.6	173.6
aD4	173.6	173.6	173.6	173.6	173.6	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	144.48	144.48	144.48	144.48	144.48

aD4	144.48	144.48	144.48	144.48	144.48	0
aD4	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	406	406	406	406	406
bD	406	406	406	406	406	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	403.2	403.2	403.2	403.2	403.2
bD	403.2	403.2	403.2	403.2	403.2	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	386.4	386.4	386.4	386.4	386.4
bD	386.4	386.4	386.4	386.4	386.4	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	392	392	392	392	392
bD	392	392	392	392	392	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	417.2	417.2	417.2	417.2	417.2
bD	417.2	417.2	417.2	417.2	417.2	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	434	434	434	434	434
bD	434	434	434	434	434	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	361.2	361.2	361.2	361.2	361.2
bD	361.2	361.2	361.2	361.2	361.2	0
bD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	270.67	270.67	270.67	270.67	270.67
cD	270.67	270.67	270.67	270.67	270.67	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	268.8	268.8	268.8	268.8	268.8
cD	268.8	268.8	268.8	268.8	268.8	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	257.6	257.6	257.6	257.6	257.6
cD	257.6	257.6	257.6	257.6	257.6	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	261.33	261.33	261.33	261.33	261.33
cD	261.33	261.33	261.33	261.33	261.33	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	278.13	278.13	278.13	278.13	278.13
cD	278.13	278.13	278.13	278.13	278.13	0
cD	0	0	0	0	0	0

cD	0	0	0	0	0	0
cD	0	289.33	289.33	289.33	289.33	289.33
cD	289.33	289.33	289.33	289.33	289.33	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	240.8	240.8	240.8	240.8	240.8
cD	240.8	240.8	240.8	240.8	240.8	0
cD	0	0	0	0	0	0

ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.3407	0.3407	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.3407	0.3407	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.3407	0.3407	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.3407	0.3407	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.4194	0.4194	0.4194	0.4194
ElectTariff	0.4194	0.4194	0.3407	0.3407	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.3407	0.3407	0.3407	0.3407	0.3407	0.3407
ElectTariff	0.3407	0.3407	0.3407	0.3407	0.3407	0.3407
ElectTariff	0.3407	0.3407	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.2698
ElectTariff	0.2698	0.2698	0.2698	0.2698	0.2698	0.3407
ElectTariff	0.3407	0.3407	0.3407	0.2698	0.2698	0.2698

December

[PATTERNS]
ID Multipliers

aD1	0	0	0	0	0	0
aD1	0	229.19	229.19	229.19	229.19	229.19
aD1	229.19	229.19	229.19	229.19	229.19	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	227.61	227.61	227.61	227.61	227.61
aD1	227.61	227.61	227.61	227.61	227.61	0

aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	218.13	218.13	218.13	218.13	218.13
aD1	218.13	218.13	218.13	218.13	218.13	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	221.29	221.29	221.29	221.29	221.29
aD1	221.29	221.29	221.29	221.29	221.29	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	235.52	235.52	235.52	235.52	235.52
aD1	235.52	235.52	235.52	235.52	235.52	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	245	245	245	245	245
aD1	245	245	245	245	245	0
aD1	0	0	0	0	0	0
aD1	0	0	0	0	0	0
aD1	0	203.9	203.9	203.9	203.9	203.9
aD1	203.9	203.9	203.9	203.9	203.9	0
aD1	0	0	0	0	0	0

aD2	0	0	0	0	0	0
aD2	0	137.52	137.52	137.52	137.52	137.52
aD2	137.52	137.52	137.52	137.52	137.52	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	136.57	136.57	136.57	136.57	136.57
aD2	136.57	136.57	136.57	136.57	136.57	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	130.88	130.88	130.88	130.88	130.88
aD2	130.88	130.88	130.88	130.88	130.88	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	132.77	132.77	132.77	132.77	132.77
aD2	132.77	132.77	132.77	132.77	132.77	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	141.31	141.31	141.31	141.31	141.31
aD2	141.31	141.31	141.31	141.31	141.31	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	147	147	147	147	147
aD2	147	147	147	147	147	0
aD2	0	0	0	0	0	0
aD2	0	0	0	0	0	0
aD2	0	122.34	122.34	122.34	122.34	122.34
aD2	122.34	122.34	122.34	122.34	122.34	0
aD2	0	0	0	0	0	0

aD3	0	0	0	0	0	0

aD3	0	183.35	183.35	183.35	183.35	183.35
aD3	183.35	183.35	183.35	183.35	183.35	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	182.09	182.09	182.09	182.09	182.09
aD3	182.09	182.09	182.09	182.09	182.09	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	174.5	174.5	174.5	174.5	174.5
aD3	174.5	174.5	174.5	174.5	174.5	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	177.03	177.03	177.03	177.03	177.03
aD3	177.03	177.03	177.03	177.03	177.03	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	188.41	188.41	188.41	188.41	188.41
aD3	188.41	188.41	188.41	188.41	188.41	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	196	196	196	196	196
aD3	196	196	196	196	196	0
aD3	0	0	0	0	0	0
aD3	0	0	0	0	0	0
aD3	0	163.12	163.12	163.12	163.12	163.12
aD3	163.12	163.12	163.12	163.12	163.12	0
aD3	0	0	0	0	0	0

aD4	0	0	0	0	0	0
aD4	0	137.52	137.52	137.52	137.52	137.52
aD4	137.52	137.52	137.52	137.52	137.52	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	136.57	136.57	136.57	136.57	136.57
aD4	136.57	136.57	136.57	136.57	136.57	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	130.88	130.88	130.88	130.88	130.88
aD4	130.88	130.88	130.88	130.88	130.88	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	132.77	132.77	132.77	132.77	132.77
aD4	132.77	132.77	132.77	132.77	132.77	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	141.31	141.31	141.31	141.31	141.31
aD4	141.31	141.31	141.31	141.31	141.31	0
aD4	0	0	0	0	0	0
aD4	0	0	0	0	0	0
aD4	0	147	147	147	147	147
aD4	147	147	147	147	147	0
aD4	0	0	0	0	0	0

aD4	0	0	0	0	0	0
aD4	0	122.34	122.34	122.34	122.34	122.34
aD4	122.34	122.34	122.34	122.34	122.34	0
aD4	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	343.79	343.79	343.79	343.79	343.79
bD	343.79	343.79	343.79	343.79	343.79	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	341.42	341.42	341.42	341.42	341.42
bD	341.42	341.42	341.42	341.42	341.42	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	327.19	327.19	327.19	327.19	327.19
bD	327.19	327.19	327.19	327.19	327.19	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	331.94	331.94	331.94	331.94	331.94
bD	331.94	331.94	331.94	331.94	331.94	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	353.27	353.27	353.27	353.27	353.27
bD	353.27	353.27	353.27	353.27	353.27	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	367.5	367.5	367.5	367.5	367.5
bD	367.5	367.5	367.5	367.5	367.5	0
bD	0	0	0	0	0	0
bD	0	0	0	0	0	0
bD	0	305.85	305.85	305.85	305.85	305.85
bD	305.85	305.85	305.85	305.85	305.85	0
bD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	229.19	229.19	229.19	229.19	229.19
cD	229.19	229.19	229.19	229.19	229.19	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	227.61	227.61	227.61	227.61	227.61
cD	227.61	227.61	227.61	227.61	227.61	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	218.13	218.13	218.13	218.13	218.13
cD	218.13	218.13	218.13	218.13	218.13	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	221.29	221.29	221.29	221.29	221.29
cD	221.29	221.29	221.29	221.29	221.29	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	235.52	235.52	235.52	235.52	235.52

cD	235.52	235.52	235.52	235.52	235.52	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	245	245	245	245	245
cD	245	245	245	245	245	0
cD	0	0	0	0	0	0
cD	0	0	0	0	0	0
cD	0	203.9	203.9	203.9	203.9	203.9
cD	203.9	203.9	203.9	203.9	203.9	0
cD	0	0	0	0	0	0

ElectTariff	0.3051	0.3051	0.3051	0.3051	0.3051	0.3051
ElectTariff	0.5347	0.5347	0.3051	0.3051	0.3051	0.3051
ElectTariff	0.3051	0.3051	0.3051	0.3051	0.9125	0.9125
ElectTariff	0.9125	0.9125	0.9125	0.9125	0.3051	0.3051
ElectTariff	0.3051	0.3051	0.3051	0.3051	0.3051	0.3051
ElectTariff	0.5347	0.5347	0.3051	0.3051	0.3051	0.3051
ElectTariff	0.3051	0.3051	0.3051	0.3051	0.9125	0.9125
ElectTariff	0.9125	0.9125	0.9125	0.9125	0.3051	0.3051
ElectTariff	0.3051	0.3051	0.3051	0.3051	0.3051	0.3051
ElectTariff	0.5347	0.5347	0.3051	0.3051	0.3051	0.3051
ElectTariff	0.3051	0.3051	0.3051	0.3051	0.9125	0.9125
ElectTariff	0.9125	0.9125	0.9125	0.9125	0.3051	0.3051
ElectTariff	0.3051	0.3051	0.3051	0.3051	0.3051	0.3051
ElectTariff	0.5347	0.5347	0.3051	0.3051	0.3051	0.3051
ElectTariff	0.3051	0.3051	0.3051	0.3051	0.9125	0.9125
ElectTariff	0.9125	0.9125	0.9125	0.9125	0.3051	0.3051
ElectTariff	0.3051	0.3051	0.3051	0.3051	0.3051	0.3051
ElectTariff	0.5347	0.5347	0.3051	0.3051	0.3051	0.3051
ElectTariff	0.3051	0.3051	0.3051	0.3051	0.9125	0.9125
ElectTariff	0.9125	0.9125	0.9125	0.9125	0.3051	0.3051
ElectTariff	0.3051	0.3051	0.3051	0.3051	0.3051	0.3051
ElectTariff	0.3051	0.3051	0.3051	0.3051	0.3051	0.3051
ElectTariff	0.3051	0.3051	0.3051	0.3051	0.5347	0.5347
ElectTariff	0.5347	0.5347	0.3051	0.3051	0.3051	0.3051
ElectTariff	0.3051	0.3051	0.3051	0.3051	0.3051	0.3051
ElectTariff	0.3051	0.3051	0.3051	0.3051	0.3051	0.3051
ElectTariff	0.3051	0.3051	0.3051	0.3051	0.3051	0.9125
ElectTariff	0.9125	0.5347	0.5347	0.3051	0.3051	0.3051

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