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Development of optimal policies for the regulation of Lake Como

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"Vola basso oggi per volare alto domani"

Martina

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CHAPTER 1

INTRODUCTION

An optimized use of the water resources is nowadays recognized as a major issue for the nations. In several countries, a significant share of the produced energy is supported by hydropower plants as well as agriculture is sustained by irrigation. In real life, most of the water resources optimization problems involve conflicting objectives, for which there is no efficient method for finding multiple trade-off optimal solutions. This concept is extensively shown by [Loucks and Van Beek, 2005], in their work it's analyzed the planning and management of water resources systems in general. The authors expose through examples many well-known methods and models on the subject, analysing pros and cons of each one of them. Most of the reservoir systems serve multiple purposes and they are multi-objective in nature. To optimize such a complex reservoir system, the dynamic programming (DP), linear programming (LP) and non-linear programming (NLP) have been widely applied in the past [William, 1985]. However, when DP is applied to a multi-reservoir system, it involves a major problem of the curse of dimensionality, with increase in the number of state variables. The techniques like LP and NLP have essential approximation problems in dealing with discontinuous, non-differentiable, non-convex multi-objective functions. The classical optimization methods such as DP, LP, and NLP are not appropriate to multi-objective optimization, because these methods use a point-by-point search approach, and the outcome for which is a single optimal solution.

The problem becomes challenging when the objectives are of conflict to each other, that is the optimal solution of an objective function is different from that of the other. In solving such problems, with or without the presence of constraints, these problems give rise to a set of trade-off optimal solutions, popularly known as Pareto-optimal solutions [Lee, 2012].

The drawbacks (efficiency and accuracy) of existing numerical methods have encouraged researchers to rely on Metaheuristic Algorithms based on simulations and nature inspired methods to solve engineering optimization problems. Metaheuristic algorithms commonly operate by combining rules and randomness to imitate natural phenomena [Eskandar et al., 2012]. In computer science and mathematical optimization, a metaheuristic is a higher-level procedure or heuristic designed to find, generate, or select a heuristic (partial search algorithm) that may provide a sufficiently good solution to an optimization problem, especially with incomplete or imperfect information or limited computation capacity.

During the past two decades Metaheuristic Methods, have been developed and widely used to extract optimal operation rules in reservoir systems. Due to the multiplicity in solutions, water management problems were proposed again to be solved suitably using Evolutionary Algorithms (EA) which use a population approach in its search procedure [Kalyanmoy, 2001]. In their paper [Reddy and Kumar, 2006] present a Multi-objective Evolutionary Algorithm

(MOEA) to derive a set of optimal operation policies for a multipurpose reservoir system also [Scola et al., 2014] make use of an evolutionary multi-objective optimization approach for the study of multiple water usages in multiple interlinked reservoirs, including both power generation objectives and other objectives not related to energy generation. Among major studies in this field, [East and Hall, 1994] used Genetic Algorithm (GA) for solving a four-reservoir problem and illustrated that significant potential exists in GA for optimizing reservoir systems; [Oliveira and Loucks, 1997] used GA to extract rule curves of a single-reservoir system and suggested it as an efficient method to achieve the optimal policy of a reservoir system operation, many other authors like [Wardlaw and Sharif, 1999] and [Chang et al., 2005] analyzed how to used GA for the optimization of reservoirs operating rules.

Although great progress has been made in the last 40 years, efficient operation of water reservoir systems still remains a very active research area. The combination of multiple water uses, non-linearities in the model and in the objectives, strong uncertainties in inputs and high dimensional state make the problem challenging.

In a strict Control Theory perspective, recent and significant advances in designing management policies for water reservoir networks, under economic, social and environmental constraints are reviewed in [Castelletti et al., 2008] paper. The focus is mainly on the implications that the nature of the storage systems (economic, social and environmental constraints) has on the formulation and solution of the mathematical control problem. It's interesting to cite a new management model, called "model pre-emption" strategy, treated by [Asadzadeh et al., 2014] and developed for increasing the efficiency of solving multi-objective optimization problems, by avoiding the full evaluation of the solution. This strategy is specifically applied to optimize a new regulation plan for Lake Superior for mitigating the extreme water levels and increase the total regulation benefits.

For what concerns Lake Como, one of the greatest lakes of Italy situated in Lombardia region, many studies and projects were made to find optimal policies for regulation. The very first important hydraulic study about Lake Como was done around 1950 by Eng. D. Citrini, who in 1972 become also Consorzio dell'Adda president. Since then various studies were done about lake floods and regulation work by Consorzio dell'Adda: [Malusardi and Moisello, 2003] reporting maximum flow rates, [Moisello and Vullo, 2010] comparing maximum levels in natural and regulated regimes,[Bertoli and Barbero, 1998] and [RegioneLombardia, 1996] about alpine reservoir regulation and their influence on MIF.

One of the most important optimization projects in this application field is exposed in the work of [Soncini-sessa et al., 1999], where the authors describe functioning and development of TwoLe software tool, for planning and management of water reservoir networks through a MODSS (Multiple Objective Decision Support Systems). Further applications of TwoLe program on Lake Como were done by Electronics Department of Politecnico di Milano, in particular by Prof. Soncini-Sessa and Prof. Castelletti, of particular importance is the report by [Amodeo et al., 2007], that analyses interaction between hydropower and irrigation system. Again [Castelletti et al., 2010] used Lake Como water system as study site to infer general guidelines on the appropriate setting for Fitted Q-Iteration Algorithm parameters and to demonstrate the advantages of the approach in terms of accuracy and computational effectiveness compared to traditional Stochastic Dynamic Programming.

1.1 Objective of the thesis

Lake Como is a regulated natural lake, the regulation device is Olginate's dam located at the end of the eastern branch and managed by Consorzio dell'Adda. The regulation of the lake is not easy and always faces problems of various nature. The limited range for acceptable water levels, the location in pre-Alps area and downstream water demands from different users make multi-objective optimization problem even more complex to be managed.

The regulation of Lake Como has three main purposes:

1. Satisfy downstream irrigation demand
2. Satisfy downstream hydropower plants demand
3. Prevent flooding of the city of Como

At the same time some constraints are imposed:

- Don't exceed upper and lower bound for lake level
- Guarantee downstream Minimum Instream Flow

The maximum outflow releasable is of the order of $900-1000 m^3/s$ when the lake level is already +250 cm at Malgrate hydrometer (over concession boundaries), for concession agreements, dam's gates can open and close varying outflow at most of $30 m^3/s$ every three hours, this means that opening maneuvers are always constrained and quite slow.

Optimization models and decision making algorithms work well with a fixed number of decision variables and parameters, they can find efficiently and quite precisely optimal solutions for a multi-objective problem in a mathematical sense. This type of "black box" algorithms reduces a very complex physical body, like a water catchment and its internal phenomena, to a series of equations and constraints to be solved, this in one side is a very functional and unambiguous way of proceeding to find the "best" solution; but in the other side doesn't take into account many other factors like natural and unpredictable phenomena, internal agreements between users, decision maker information that can't be expressed by equations. Solutions and outputs can be different, with regard to models which aim is to find an optimal policy for a reservoir regulation, a typical decision variable as output can be the delivered outflow.

Generally during one year at most 100 maneuvers are made, remembering that for instance to pass from $90 m^3/s$ to $200 m^3/s$, at least four maneuvers are necessary. Numerical solutions that aim to maximize/minimize a multi-objective function, they typically correct the deliver outflow frequently to be as close as possible to the maximum/minimum.
Moreover, optimization model using weather forecasts can vary their solution any minute, according to the changes in weather patterns.

All this makes most of the theoretically optimal policies hardly applicable to the real case, or even not applicable at all. For the specific case of Lake Como, the volume available for regulation is very small and flow rates from concession contracts are in general unsatisfiable due to low water availability. The objective of this work is to create a tool useful for the management authority, Consorzio dell'Adda, for planning the regulation of Lake Como in a quite long period. The program suggests possible regulation policies to the decision maker who can make changes and suggestion to the program itself; the tool also relies on the experience of the management authority that is fundamental for a right management of the basin and gives him an overall control of Adda river system.

1.2 Outline

The choice of an optimal regulation policy is a long process not circumscribed only to the lake regulation itself. The developing requires a good understanding of the whole catchment basin of interest, of the geographical location, of constraints and users needs, any decision made upstream for lake regulation influences also all the downstream river system, so any choice has to be calibrated observing the complete picture. For these reasons it's also added to the tool for lake regulation, a simulator representing downstream river system and flow rate diversions.

The present thesis work is organized in eight Chapters, as follows:

Chapter 1, an introductory part about muti-objective optimization methods and their application in water management and reservoir regulation. The chapter ends with a focus on Lake Como case and a general presentation of the tool for planning optimal regulation policies developed in this work.

Chapter 2, of theoretical nature, is a digression defining the modelling approach in general and the role of models in water resources planning and management projects. Optimization and simulation methods are compared and described in their main features, as well as evolutionary algorithms are shortly presented in the last part.

Chapter 3, an overview of the case of study, that is Lake Como and river Adda water catchment. The chapter contains information regarding geographic contextualization, lake regulation and inflows to the reservoir. The downstream Lake Como water catchment is then described, giving a particular look at irrigation channels, hydropower plants and MIF.

Chapter 4, presents the modeling of Adda river system. River Adda is approximated as a series of hydraulic segments in a row, where each segment is solved by continuity equation. The model of Adda makes possible to simulate different policies for water allocation, starting from delivered outflow and computing a plausible hydropower production for all the system.

Chapter 5, deals with downstream water allocation. The first part analyses irrigation users demand and current Consorzio del'Adda water allocation policy. The second one describes the use of Dynamic Programming Network (DPN), an optimization method, for solving river Adda water management problem and finding an optimal policy for downstream water supply.

Chapter 6, introduces the very optimization model, that is the tool for planning Lake Como regulation policy starting from projections for future incoming volumes. Three different approaches are compared for inflows forecasts: empirical, probabilistic a priori and probabilistic a posteriori; lastly the choice of the regulation policy is discussed.

Chapter 7, of applicative nature, presents the implementation of the chosen regulation policy to the real case. Since an effective application is clearly impossible for practical reasons, the management decisions made are tested thanks to a simulation of the variation of lake level, starting from inflows historical data, and thanks to the model of downstream Adda river system.

Chapter 8, a summary of the main results of the work plus suggestions for future work and lessons learnt from the procedures. Remarks regarding the main features of the optimization tool for Lake Como regulation and the approximated model of Adda river system.

CHAPTER 2

WATER RESOURCE SYSTEM MODELS

A single or multiple water resource system is composed of various natural components including natural lakes, rivers, wetlands, aquifers, and engineered components such as reservoirs, channels, tunnels, pipelines, hydropower plants, pumping stations and water consumption sites. It is represented as a network of nodes interconnected, and it operates to supply water for agricultural, industrial and municipal needs, recreation, navigation and ecological issues. The different components interact each other and with the surrounding environment and can be influenced by human policies. Planning, designing and managing water resources systems today inevitably involve impact prediction. Impact prediction involves modelling. Model structure, input data, objectives and other assumptions related to how the real system functions or will behave under alternative infrastructure designs and management policies or practices may be controversial or uncertain. As useful as they may or may not be, the results of any quantitative analysis are always only a part, but an important part, of the information that should be considered by those involved in the overall planning and management decision-making process [Loucks and Van Beek, 2005].

The presence of many stakeholders with different objectives increase the necessity of these tools, because they can improve the efficiency, allowing a rapid and comprehensive evaluations of the alternatives. The integrated optimal allocation model for complex system of water resources management consists of following three separately modules [Zhou et al., 2015]:

- Agent-based module: for revealing the various characters and their role in a complex water system, such as administrative agent, water supply agent (reservoir and hydropower station agent), water user agent (industrial production and domestic water agent, agricultural production water agent, hydropower station agent, ecological water agent...)

- Optimal module: for deriving decision set of water resources allocation using multi-objective algorithms. Considering constraints and multi-objective functions.

- Evaluation module: for evaluating the efficiency of the optimal module and selecting the optimal water resources allocation scheme using project pursuit method.

2.1 MODEL COMPONENTS AND DEVELOPMENT

The model describes, in mathematical terms, the system being analysed and the conditions that the system has to satisfy, these conditions are often called *constraints*.
A mathematical model is a series of algebraic equations solving together a complex system. These equations include variables that are assumed to be known, called *parameters*, and others that are unknown to be determinated, called *decision variables*.

These decision variables can include design and operating policy variables of various water resources system components. Design variables for instance can be storage capacities of reservoirs, flow capacities of canals, capacity of hydropower plants, targets for water supply allocations and so on. Operating variables can include released water from reservoirs and the water allocation among the various users over time and space. Other decision variables can also be measures of system performance (net benefit, pollution, hydrological targets...).

Before the development of a quantitative simulation model, is useful to develop a conceptual one. Conceptual models are non-quantitative representations of a system, the overall system structure is defined but not all its elements and functional relationships.

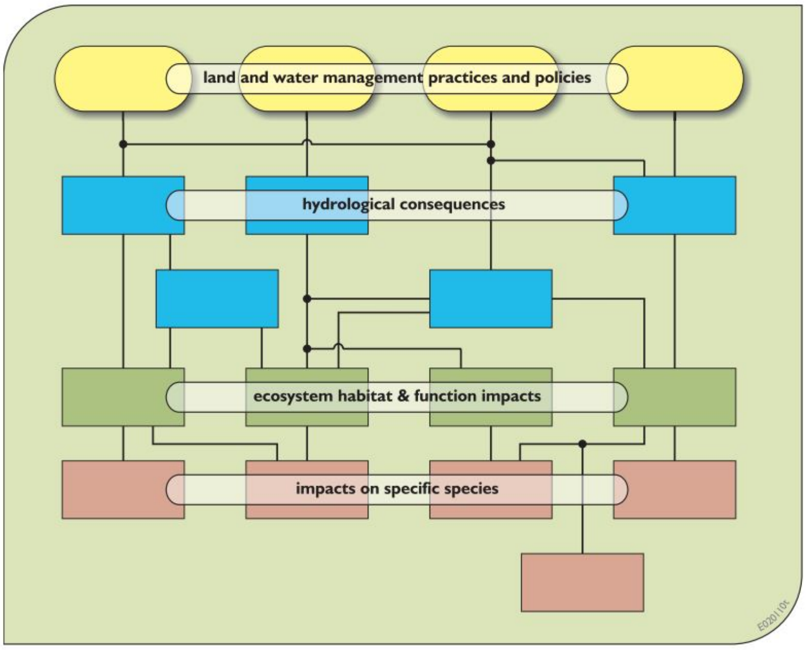


Figure 2.1: Example of conceptual model without its detail, showing the cause and effect links between management decisions and specific system impacts [Loucks and Van Beek, 2005]

Once the conceptual model has been quantified (expressed in methematical terms) it becomes a mathematical model. The values of the model's parameters need to be determined. Model calibration involves finding the best values for these parameters, it is based on comparisons of the model results with field measurements or time series. If some variables are uncertain, a sensitivity analysis can be useful to quantify them. When the model is fully developed it has to be validated, the verification process is done comparing the model results with measured observations not used in calibration step. This comparison is made to verify whether or not the model describes the system behaviour sufficiently correctly.

2.2 Plan formulation and plan selection

Solving a model means finding values of its unknown decision variables that can define a plan or a policy. Plan formulation consists in assigning particular values to each of the relevant decision variables, following a certain strategy of management. Plan selection is the process of evaluating alternative plans and selecting the one that best satisfies a particular objective or set of objectives.

The first thing to do is identify the main objectives and the possible alternatives.

Each objective is described by an *objective function* $f_i(X)$, $i = 1,2,...,k$, where X is the vector of the decision variables x_j, $j = 1,2,...,n$.

The vector X of decision variables is represented by:

$$X = [x_1, x_2, ..., x_n]^T \tag{2.1}$$

In a water resources system the objectives can be multiple and of different nature such as economic (i.e. Net Benefit), hydrological (flood control), ecological, social... Therefore, the objective functions form a vector function $f(X)$ which is defined by:

$$f(X) = [f_1(X), f_2(X), ..., f_k(X)]^T \tag{2.2}$$

There are various approaches to find the "best" plan or best set of decision-variable values.
When there is not one objective function to optimize, but many we are talking about *Multiobjective Optimization Probelm*, shortly *MOP*. More precisely, multiobjective problems (MOPs) are those problems where the goal is to optimize k objective functions simultaneously. This may involve the maximization of all k functions, the minimization of all k functions or a combination of maximization and minimization of these k functions.

"A general MOP is defined as minimizing (or maximizing) $F(X) = (f_1(X),...,f_k(X))$ *subject to* $g_i(X) \leq 0$, $i = \{1,...,m\}$, *and* $h_j(x) = 0$, $j = \{1,...,p\}$. *An MOP solution minimizes (or maximizes) the components of a vector* $F(X)$ *where* X *is a n-dimensional decision variable vector* $X = (x_1,...,x_n)$ *from some universe* Ω. *It is noted that* $g_i(X) \leq 0$ *and* $h_j(X) = 0$ *represent constraints that must be fulfilled while minimizing (or maximizing)* $F(x)$ *and* Ω *contains all possible* X *that can be used to satisfy an evaluation of* $F(X)$*"* [Coello et al., 2007]

The existence of many feasible alternative plans is a characteristic of most water resources systems planning problems. The particular feasible solution or plan that satisfies the objective function, that maximizes or minimizes it, is called *optimal*. It is the optimal solution of the mathematical model, but it may not necessarily be considered optimal by any decision-maker. What is optimal with respect to some model may not be optimal with respect to those involved in a planning or decision-making process. On the other side when there are multiple conflicting objectives it's possible to find solutions that are "non-dominated" or *Pareto-optimal* solutions, in this case the choice is given by the decision making or the policy we want to adopt [Lee, 2012].

2.2.1 Pareto-optimal Solutions

In the single-objective optimization problem, the superiority of a solution over other solutions is easily determined by comparing their objective function values. In multi-objective optimization problem, the goodness of a solution is determined by the *dominance*.
Dominance Test

Given two different solutions x_1 and x_2, x_1 dominates x_2 if

- Solution x_1 is no worse than x_2 in all objectives
- Solution x_2 is strictly better than x_2 in at least one objective

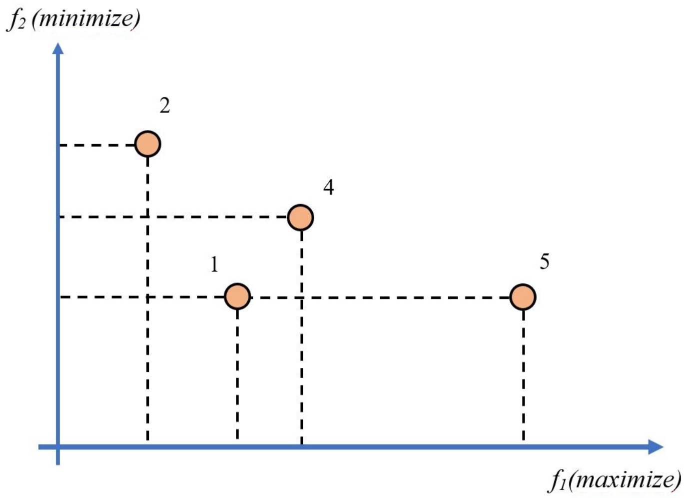


Figure 2.2: Example Dominance Test

In Fig.2.2 for 1 vs 2: 1 dominates 2; for 1 vs 5: 5 dominates 1; for 1 vs 4: neither solution dominates.

Given a set of solutions, the *non-dominated solution set* is a set of all the solutions that are not dominated by any member of the solution set. The non-dominated set of the entire feasible decision space is called the *Pareto-optimal set,* the boundary defined by the set of all point mapped from the Pareto optimal set is called the *Pareto-optimal front* [Kalyanmoy, 2001].

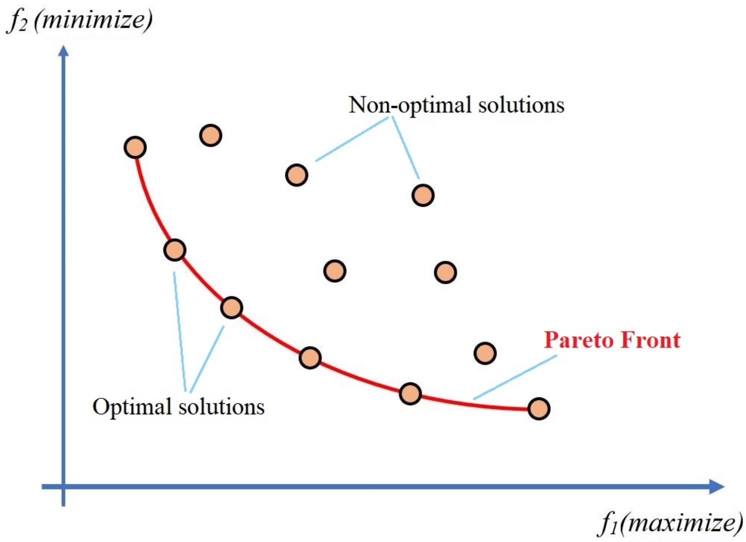


Figure 2.3: Graphical Depiction of Pareto Optimal Solution

The main goal of multi-objective optimization is finding a set of solutions as close as possible to Pareto-optimal front, and as diverse as possible from each other. The final choice will be taken by the decision maker, based on its experience and on objectives considered a priority.

2.3 Modelling Methods: Simulation and Optimization

Two different approaches can be used for the development of a generalized model: optimization and simulation modelling approach. The main difference is that simulation models predict system performance for a user-specified set of variable values, while optimization models search for an optimal solution, with the maximization or minimization of the objective function. The term optimization is used synonymously with mathematical programming to refer to a mathematical algorithm that computes a set of decision variable values which minimize or maximize an objective function subject to constraints [Kim and Wurbs, 2011].

Simulation models have the advantage that they can include much longer time series of hydrological, economic and environmental data, because they are not limited of many of the assumptions incorporated into optimization models. Compared to optimization models, a more detailed and realistic representation of the complex water resource system is possible. As a consequence, resulting outputs can better identify the variations of hydrological, environmental and economic impacts over time. The problem concerning water resource systems is the presence of many variables, with an infinite combination of feasible values. The difficulty with using simulation alone for analysing multiple alterna- tives occurs when there are many alternative, and potentially attractive, feasible solutions or plans and not enough time or resources to simulate them all, for this reason simulation works when there are only few alternatives to be evaluated.

The advantages of optimization model concern a more prescriptive analysis, determining the plan or policy that should be adopted to satisfy the decision criteria. It provides a more systematic and efficient computational algorithm, various assumptions are taken in order to find in an easier way the best solution. Moreover, the use of optimization eliminates many line of code (in particular if we have a very complex water catchment with many elements) and increases the readability of the model. One criticism of math programming models concerns the difficulty to include all the operations and constraints in the mathematical framework; in particular optimization models need explicit expressions of objectives that sometimes are not so simple, for instance when objectives concern environmental and social impacts. For water resources planning and management, it is often advantageous to use both optimization and simulation modelling; firstly optimization to reduce the number of alternatives, then simulation, removing some assumptions, in order to find a more "real" situation.

2.3.1 Simulation models

Simulations models can be:

- STATISTICAL based on data (field measurements).
- PROCESS-ORIENTED based on knowledge of the fundamental processes that are taking place.
- Combination of the previous two.

The relationship between the input and the output variable values is derived by calibrating a black-box or statistical model with a predefined structure unrelated to the actual natural processes taking place. Once calibrated, the model can be used to estimate the output variable values as long as the input variable values are within the range of those used to calibrate the model [Loucks and Van Beek, 2005].

2.3.2 OPTIMIZATION MODELS

Optimization methods are designed to provide the best values of system design and operating policy variables, so that can lead to the highest level of system performance. The procedure (or algorithm) most appropriate for solving any particular optimization model depends in part on the particular mathematical structure of the model. There is no single universal solution procedure that will efficiently solve all optimization models.

- NON-LINEAR OPTIMIZATION MODELS (mathematical non linear programming NLP) [Floudas, 1995] based on calculus to find the maximum (or minimum) of the objective function. Their practical application it's possible if the functions are continuous and concave for maximization (or convex for minimization), otherwise the solution can be difficult or leading to only local optimal solution. Examples of calculus-based methods are *Hill-Climbing Method* and *Lagrange Multipliers*.
- DYNAMIC PROGRAMMING [Böhme and Frank, 2017] [Loucks and Van Beek, 2005] divides the original optimization problem into a set of smaller optimization problems (discretization) each of which needs to be solved before the overall optimum solution to the original problem can be identified. These methods find an approximated best solution, then simulation it's needed to refine the solution.Examples of dynamic programming are *Dynamic Programming Networks (Backward-Moving and Froward-Moving Solutions)* and *Numerical Solutions*.
- LINEAR PROGRAMMING [Nocedal and Wright, 2006] is the most widely used of all optimization tools, there are many computer programs and software available, but it requires that both the objective function and the constraints are linear. Often, the situations they model are actually nonlinear, but linear programming is appealing because of the advanced state of the software, guaranteed convergence to a global minimum.

2.4 EVOLUTIONARY ALGORITHMS (EAS)

Most optimization and simulation models used for water resources planning and management describe, in mathematical terms, the interactions and processes that take place among the various components of the system. These mechanistically or *process-based models* usually contain parameters whose values are determined from observed data during model calibration.

These types of models are contrasted to what are typically called "black-box" models, or statistical models, *data-based models* . Statistical models do not describe physical processes. They attempt to convert inputs (e.g., rainfall, inflows to a reservoir) to outputs (e.g., runoff, reservoir releases) using any mathematical equation or expression that does the job.

Regression equations, to estimate parameters starting from observed inputs and outputs, are examples of data-based modelling methods. Other examples of such methods are based on evolutionary principles and concepts. These are a class of probabilistic search procedures known as *Evolutionary Algorithms* (EAs). Such algorithms include Genetic Algorithms (GAs), Genetic or Evolutionary Programming (GP or EP) and Evolutionary Strategy (ES). Evolutionary Multiobjective Optimization (EMO), refers to the use of evolutionary algorithms of any sort to solve multiobjective optimization problems [Coello et al., 2007].

Evolutionary algorithms are the algorithms that are based on the evolution of the species; in general they are based on the main evolutionary theory of Charles Darwin. The way the evolutionary mechanisms are implemented varies considerably; however, the basic idea behind all these variations is similar. EAs work on a population of potential solutions, which use the survival of the proper principle to construct consecutively better approximations to a solution. A new approximation set is produced by the selection of an individual process, the basis of the fitness level in the problem field and reproduction with use of variation operators in each EA generation.

EAs are a population set-based optimization which use bio-inspired mechanisms, including mutation, crossover, natural selection, and survival of the fittest to refine a set of solution candidates iteratively. The basic generic EA first initializes a population of solution candidates (initial population), then the following three procedures are repeated, which are: (1) assesses the population individual's fitness, (2) uses this fitness information to breed a new population of children, and (3) combines the solutions for the parents and children in some way to form a new generation of the population, and the cycle of the process continues iteratively [Siddhartha et al., 2017].

This type of "black box" algorithm works very well for many problems, finding the optimal solution precisely; but, as explained in Chapter 1, often the mathematically optimal solution doesn't correspond to a feasible policy in reality, especially for what concerns water management where the choice of regulation policy is made thanks to experience of the decision maker and under influence of some information that can't be expressed by an equation.

CHAPTER 3

THE CASE OF STUDY: RIVER ADDA WATER CATCHMENT

Consorzio dell'Adda is a non-economic public authority (D.P.R. 532 of 1st April 1978, supervised by Ministero dei Lavori Pubblici and instituted with R.D. 2010 of 21th November 1938). The Consorzio is composed of all the agricultural and industrial users of the Adda's waters, it has the role of ensuring construction, maintenance and exercise of the Olginate's dam and managing the allocation of the river water flow between the users. The building of the dam started in 1941 and finished in 1944, the real regulation started in 1946 [RegioneLombardia, 1996].

In addiction to the hydropower production, the so called "acque nuove" (new waters) are used to increase the water availability for big irrigation districts that extend for over 136 thousand hectares and comprehend territories served by Naviglio della Martesana (North of Milan), ditch Vailata (in Bergamo's area), canal Muzza (in Lodi's area), canal Retorto, ditch Rivoltana and canal Vacchelli (in Crema's and Cremona's area).

The main problems connected to the regulation of the Lake are the phase shift of the inflows due to the regulation of the alpine reservoirs (in Valtellina Valley) and subsidence phenomena in the city of Como (in the southwestern arm of the lake).

3.1 GEOGRAPHIC FRAMEWORK

Lake Como, called Lario, is one of three main italian lakes, with Lake Maggiore and Lake Garda. It's located in Lombardy at the altitude of 197 m a.s.l. and its surface is $145 km^2$, with a volume about $23.37 km^3$, this makes the lake a fundamental water resource for the region.

The Lario is a part of a wider hydraulic system which is the River Adda water catchment. The Adda's souce is near the "Monte del Ferro" at 2150 m a.s.l. in the Raetian Alps. After descending the Fraele Valley, feeding the artificial Lakes of Cancano, gathers with the waters from River Braulio, and increases its volume, at the beginning very limited, near the town of Bormio. Thence it flows first southwest, then due west, through the fertile Valtellina, along which it receives water contribute from some other tributaries, until it falls into the Lake Como. This first half path of Adda makes it the very only one big river of Northern Italy to flow from East to West.

In the southeastern or Lecco arm of the lake it's situated the regulation device (a little dam), known as "Traversa di Olginate", between the little lakes of Garlate and Olginate; at this section Adda's river basin extends for $4552 km^2$.

The river flows through the plain of Lombardy "Pianura Padana" among the province of Lecco, Bergamo, Monza-Brianza, Lodi, Cremona until falls into the River Po at Castelnuovo Bocca d'Adda (LO) at 35 m a.s.l.. In this last part it's important to highlight the confluence with River Brembo, near Crespi d'Adda (BG) and with River Serio, near Gombito (CR).

The total length of River Adda is 313 km with a medium discharge of $187 m^3/s$. The whole catchment area is about $7927 km^2$, making it the 11% of total Po Basin's surface; River Adda Basin is for the 94% in Italian territory and for the remaining 6% in Swiss one, with 79% in mountainous area and 21% in plain area.

Figure 3.1: Adda River Basin [Water2Adapt and FEEM, 2011]

3.2 LAKE REGULATION

The regulation band of Lake Como, indicated in concession specifications of 12th January 1942, is of 170cm, with a storage capacity of 246.5 mln m^3 with principal irrigation aim in summer and hydropower production in winter. The storable volume is about a twentieth of the total water transiting the lake in one year, and now it has been reduced because of the subsidence phenomena in Piazza Cavour in Como [Bertoli and Barbero, 1998].

The regulation work of the Consorzio mainly consists in holding the water during floods and deliver it in drought periods affected by water scarcity. So, there is also an important lamination effect during floods, useful for riparian citizens. The water kept and provided to the users it's called "acqua nuova" (new water), since before the regulation it wasn't available.

The main structure is a restraint dam, located upstream the bridge Olginate-Calolziocorte, between the Lake of Garlate and the Lake of Olginate. The dam, made of compressed air foundations, is 150m long and it's subdivided into 8 spans of 14m each one with a total height of the threshold of 195 m a.s.l. The spans are closed by flat sluice-gates with rolls 4m high, that can be handled both manually and electrically. On the left side it's located an artificial valley for navigation, now it's occasionally used as fishway towards Lake Como. It was also fixed Adda's river bed till the Robbiate's dam for a total length of 5.5 km, lowering and locally expanding the course of the river in order to increase the flow discharge capacity [Di Patti et al., 2013].

Figure 3.2: Olginate's dam

3.2.1 Regulation boundaries

The regulation band in concession to the Consorzio dell'Adda, indicated in the concession disciplinary of 21/01/1942 it's between -50cm to +120cm referred to the Fortilizio's hydrometer.

Since the Fortilizio's hydrometer is subjected to the influence of river Adda at the exit of the Lake at Lecco, is common to use as reference system the Malgrate's hydrometer,197.37 m a.s.l., to which correspond the regulation boundaries of $-40cm$ and $+130cm$.
These two limits aren't simply formal, but are based on real needs of the riparian citizens.

The level +130cm corresponds to the overflow threshold of Piazza Cavour. With the passing of the years the city of Como has been subjected to subsidence phenomena, and nowadays flooding events can happen even for lower levels of the lake, for this reason the upper boundary has been moved to $+120cm$. A new system of barriers and sluice-gates has been designed to prevent flooding, the beginning of the construction works is planned for years 2021/2022.

The level -40cm, regards different aspects:

- it's necessary to keep a minimum level for the use of docks and piers and to guarantee the regular navigation in the lake
- an excessive excursion of the level (especially in a short span of time) leads to geotechnical problems, like erosion and breaking of lake banks. The moving up and down of the level creates a continuous changing of water pressure upon the walls and the structures lying on the shores, this can cause instability and collapses.
- a too low level causes problems to the fauna and the fishing life, in particular for what concerns the reproduction of some species of fish like "alborella" (bleak fish) and "lavarello" (common whitefish).

Table 3.1: Regulation boundaries, Malgrate hydrometer

Upper-bound	+ 120 cm
Lower-bound	-40 cm

The problems before explained start already when the lake is at a level of -20cm, followed by complaints from the riparian population, this leads to even less freedom for regulation excursion.

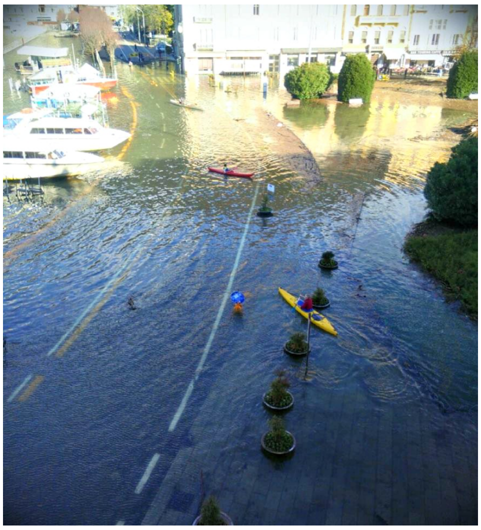


Figure 3.3: Lake level +150cm

Figure 3.4: Lake level -7cm

It is noticed that with a natural regime the lake would reach even higher levels and for longer periods. The managing body is applying its regulation work keeping the water level within the boundaries, avoiding in part the problems that naturally would occur, in favor of the riparian citizens and the city of Como.

Anno	Regime naturale			Regime semiregolato			Regime regolato		
	num. allag.	num. giorni	h_{max} [cm]	num. allag.	num. giorni	h_{max} [cm]	num. allag.	num. giorni	h_{max} [cm]
1980	2	33	84,9	2	17	61,9	2	10	74,5
1981	3	50	88,5	3	42	82,9	3	15	49,7
1982	2	18	54,3	2	11	46,9	2	4	18,0
1983	1	67	185,9	1	44	165,0	1	17	87,5
1984	2	50	50,6	2	41	38,2	0	0	-
1985	4	61	76,6	2	41	50,7	0	0	-
1986	3	83	120,8	4	64	96,4	2	14	15,5
1987	3	85	219,8	5	70	187,2	5	30	143,0
1988	5	62	63,5	3	42	50,5	1	4	19,0
1989	2	21	16,5	2	5	8,0	0	0	-
1990	2	24	24,5	1	2	4,2	1	4	13,0
1991	3	49	77,1	3	29	60,3	2	5	17,0
1992	3	49	38,1	4	30	26,4	0	0	-
1993	2	49	222,8	1	48	217,0	1	34	144,0
1994	2	42	60,3	2	37	52,8	0	0	-
1995	0	0	-	0	0	-	0	0	-
1996	4	32	74,7	3	22	71,3	2	7	11,0
1997	1	33	209,2	1	29	179,2	1	13	113,5
1998	2	43	68,7	3	21	41,3	0	0	-
1999	5	56	96,5	3	41	85,4	2	7	22,5
2000	6	94	170,9	5	83	175,3	3	25	89,0

Figure 3.5: Annual number of flooding events, annual number of days of flood and annual maximum h_{max} of the difference between the level of the lake and the level at which begins the overflow in Piazza Cavour [Malusardi and Moisello, 2003]

Notice: the first column is the year, the three later are about the natural regime, then the semi-regulated regime, where aren't taken into account alpine reservoirs, and at the end regulated, the effective real situation.

3.3 INFLOWS

River Adda is located a in a complex hydrographic basin, since it stretches from north to south crossing all Lombardia region. Lake Como is the connection between the upstream area, comprehending the Valtellina Valley with its alpine reservoirs and the downstream area in the Po plain with its irrigation channels and cities.

This makes the Lake and Adda the collection center of various inflows of different nature, the computation of the amount of water is quite complex since it isn't only based on precipitation but also takes into account the time of overflow, water retention from alpine reservoirs and their variable water discharges, infiltration and groundwater percolation, snow melting...

Flood events are the "dangerous" and usually unpredictable part of the inflows, they can cause serious damages downstream if not well controlled. These flood events are mainly concentrated in two periods of the year: during spring rains (to which it's necessary to add the snow melting) and autumn ones.

In a good regulation work it's always good to stay on "safe side" and keep a certain safety margin to manage this big amount of water suddenly incoming. This empty volume, called lamination volume has remain always free and never neglected during design and managing.

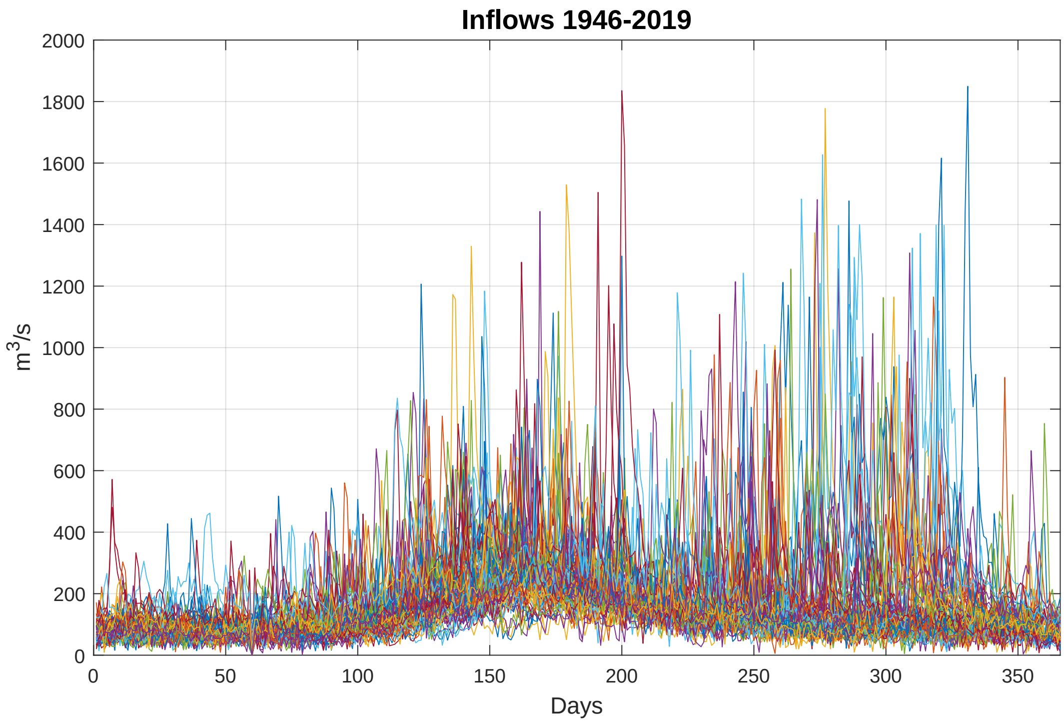


Figure 3.6: Inflows 1946-2019

It's done a deeper analysis over 10 years between 2007 and 2016 to generally understand the behavior of the inflows.

The seasonal behavior of inflows creates macro-periods within the year, during which there is a general tendency to fill or empty the reservoir.

Generally in winter the level is kept quite low: first of all there aren't a lot of precipitation and in the Alps the precipitation is for the most of the times snowy, so doesn't create inflows; second in spring it's expected a great volume of water incoming because of the spring rains and the snow melting; third in winter there are not huge irrigation needs, so the water level can be kept as much as possible on "safe side".

During April-May the level of the lake begin to increase, trying to store water with the view to the dry summer period. Starting from these months, continuing until August, the agricultural needs are very huge; in order to satisfy all of them it's necessary to discharge almost all the volume of water stored in spring.

Usually at the end of August the levels are the lowest recorded. With the arrival of the autumn rains, the level restarts to increase with an up and down trend depending on the hydraulic requests and irregular floods.

Table 3.2: Extreme inflows $[m^3/s]$

Maximum daily inflow (18/07/1987)	1836
Maximum daily inflow in 2007-2016	1323
Minimum daily inflow in 2007-2016	10.4

In Fig. 3.7 is made the mean over 15 days for the inflows, highlighting the periods in which generally the lake fills and empties, changing the water availability.

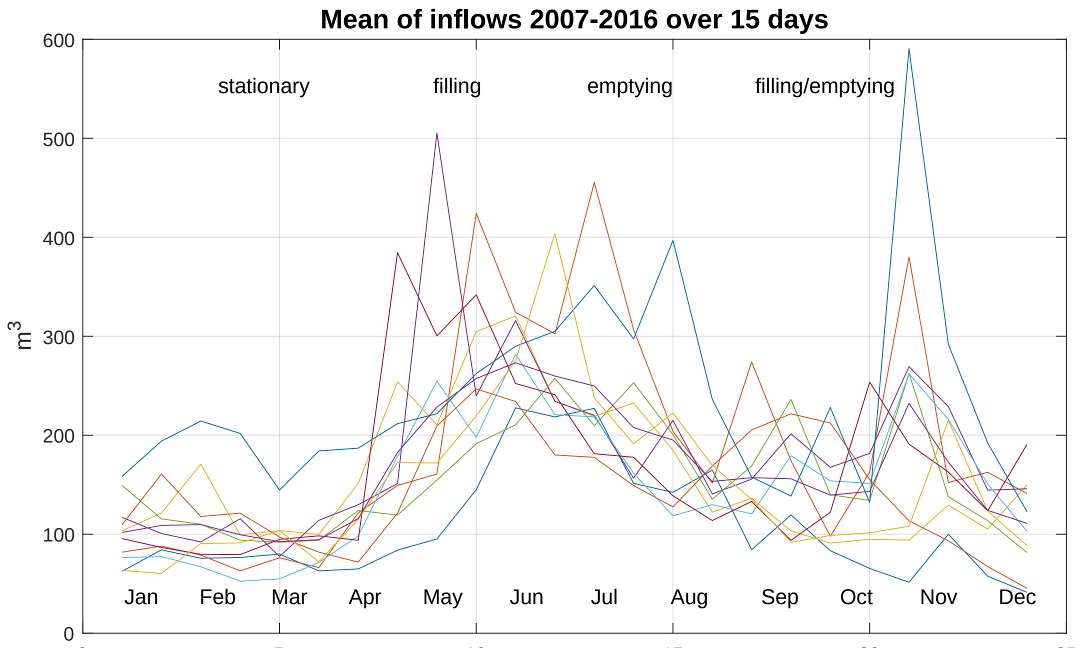


Figure 3.7: Inflows 2007-2016, mean over 15 days

Table 3.3: Monthly means $[m^3/s]$

	2007	2008	2009	2010	2011	2012	2013	2014	2015	2016	Yearly mean
Jan	73.75	85.05	112.76	108.62	131.81	76.93	90.97	176.87	136.05	61.90	105.47
Feb	76.10	71.23	137.63	103.10	102.65	60.12	79.57	208.53	119.47	90.91	104.93
Mar	72.12	71.07	100.60	93.70	92.96	63.47	96.98	166.65	89.78	87.43	93.48
Apr	73.16	135.42	200.57	137.90	120.07	138.41	226.61	196.56	92.83	143.76	146.53
May	116.51	296.67	259.08	368.17	169.71	225.32	329.71	241.93	228.92	196.48	243.25
Jun	226.07	313.37	276.95	275.92	235.93	251.23	248.24	293.91	207.22	339.62	266.85
Jul	187.46	378.57	225.64	191.04	231.89	189.20	179.63	322.73	163.55	213.86	228.36
Aug	155.59	176.99	155.95	177.94	167.05	124.44	128.14	320.20	148.85	195.28	175.04
Sep	102.04	224.15	120.47	176.85	203.35	149.94	112.92	149.54	213.66	113.58	156.65
Oct	75.82	131.41	93.79	161.65	137.31	152.56	187.18	180.29	183.87	100.18	140.41
Nov	74.81	266.15	108.52	261.07	200.07	239.04	180.18	432.47	104.97	161.54	202.88
Dec	51.46	151.47	133.97	150.80	101.38	126.07	161.37	167.80	58.17	105.45	120.79

In Tab. 3.3 are colored in green the maximum mean inflow value over each year and in red the minimum. It can noticed that the "green" values are always between the months of May and June (when there is a greater discharge from snow melting and spring rains) and in winter are placed the "red" values coincident with the periods of least liquid precipitation.

Inflows data series are taken as a certain data and will be used as historical input for testing models. Actually inflows aren't directly measured by gauges, instead they are computed backward, using continuity equation, starting from measured levels and outflows. Since Lake Como is very long, the lake level can be affected by oscillations due to wind and waves, for this reason Hortus Srl (www.laghi.net) and Progea Srl apply level smoothing models to avoid this errors and make the computation more precise before sending inflows data to Consorzio dell'Adda.

3.4 MIF Minimum In-stream Flow

"The MIF (DMV in Italian) is the runoff, in a natural river course, that has to be present downstream the hydraulic withdrawals in order to keep viable the functionality conditions and the quality of the ecosystems involved, compatibly with a well-balanced use of the water resource"(Translated from DGR VIII/2244 del 29.03.06 Regione Lombardia).

There are many methods, both theoretical and experimental, to compute the minimum in-streamflow to reach a sustainable use of riverine co-habitats. Region Lombardy uses a theoretical method based on the annual mean discharge , MIF is a fixed percentage of the natural river discharge, for Lombardy is 10%.

$$Q_{MIF} = 10\% E[Q]$$

Actually the formula is more complicated involving factors regarding the catchment area, morphology,natural characteristics, interaction between groundwater and superficial water basin and the necessity of vary the MIF over time.

For river Adda was done a big studio and experimentation in order to find the MIF in the various sections of the river, since the river course is very long and there are many withdrawals, the MIF changes from a section to one other. For each intake is determined a different MIF [ConsorzioDell'Adda, 2014].

In total are defined eleven cross sections for the control of MIF, but three of them are located in the node of canal Muzza, with a quite complex system of spillways and barriers, an approximation is made and for the rest of the work will be taken into account only the last of these group, called downstream IV spillway.

Table 3.4: Monthly MIF [m^3/s]

Intake	DMV gen	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Robbiate's Dam	15.3	16.5	9.1	9.1	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5
Paderno's Dam	11.9	9.1	9.1	9.1	11.6	14.9	16.6	16.6	9.9	9.9	9.9	16.6	9.9
Trezzo's Dam	8.4	8.4	8.4	8.4	8.4	8.4	8.4	8.4	8.4	8.4	8.4	8.4	8.4
Concesa's Dam	12.0	9.2	9.2	9.2	11.7	15.1	16.7	16.7	10	10	10	16.7	10
Sant'Anna's Dam	20.3	20.3	20.3	20.3	20.3	20.3	20.3	20.3	20.3	20.3	20.3	20.3	20.3
Rusca Dam	13.8	14.3	14.3	14.3	12.2	10.2	20.4	10.2	10.2	12.2	12.2	20.4	14.3
Retorto Dam	10.6	10.6	10.6	10.6	10.6	10.6	10.6	10.6	10.6	10.6	10.6	10.6	10.6
Fish pass Muzza	4.0	4	4	4	4	4	4	4	4	4	4	4	4
III spillway Muzza	14.8	14.8	14.8	14.8	14.8	14.8	14.8	14.8	14.8	14.8	14.8	14.8	14.8
IV spillway Muzza	21.2	21.2	21.2	21.2	21.2	21.2	21.2	21.2	21.2	21.2	21.2	21.2	21.2
Vacchelli intake	22.1	22.1	22.1	22.1	22.1	22.1	22.1	22.1	22.1	22.1	22.1	22.1	22.1

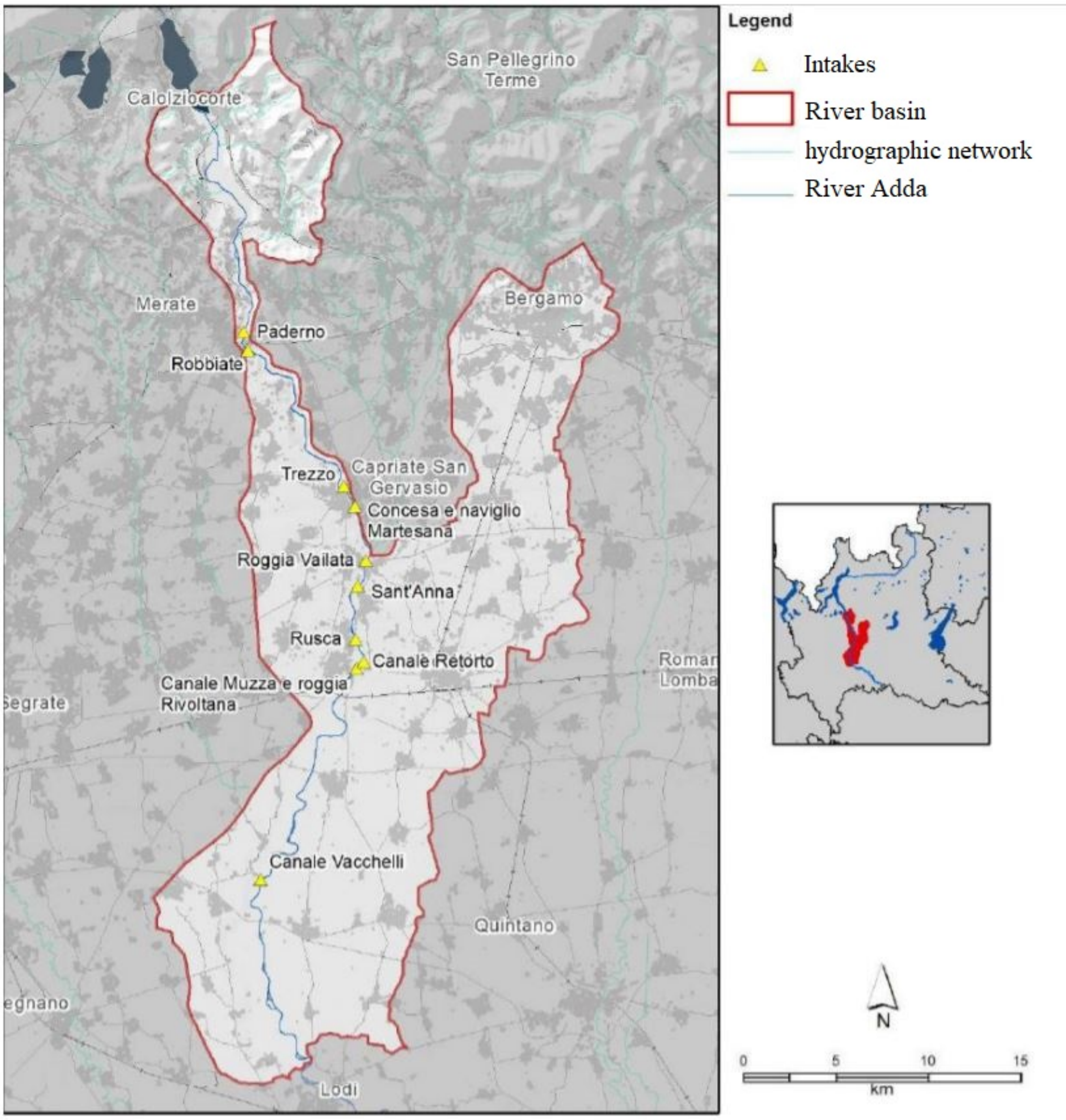


Figure 3.8: Intakes along river Adda [Maroni and Aprea, 2017]

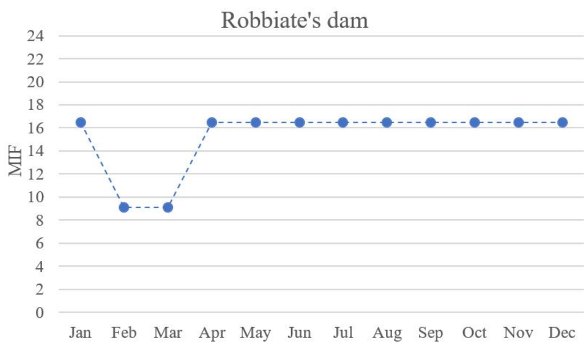


Figure 3.9: MIF Robbiate's dam

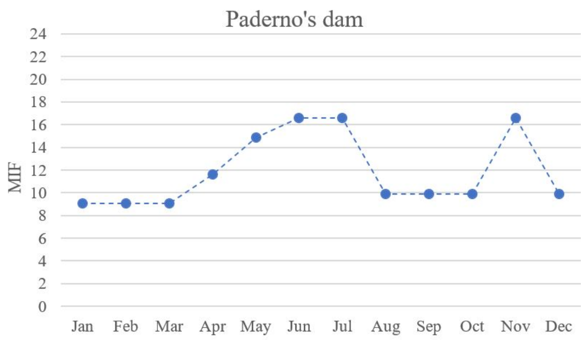


Figure 3.10: MIF Paderno's dam

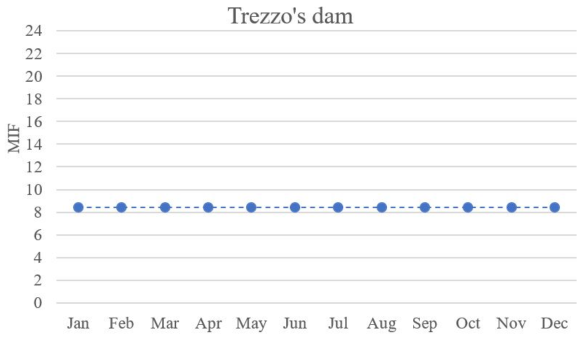


Figure 3.11: MIF Trezzo's dam

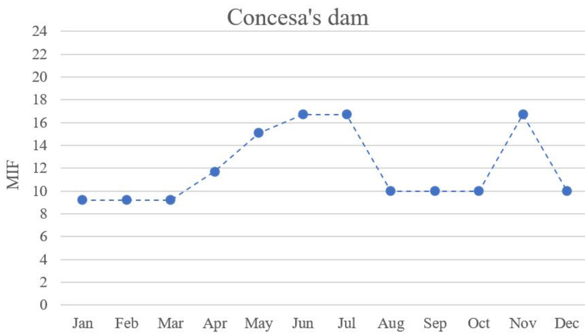


Figure 3.12: MIF Concesa's dam

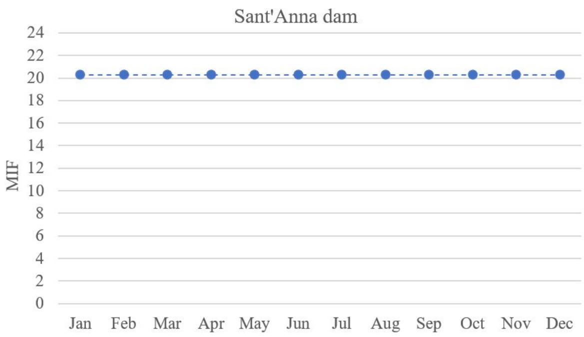


Figure 3.13: MIF Sant'Anna dam

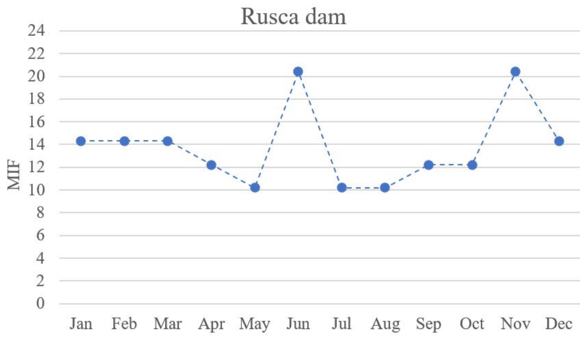


Figure 3.14: MIF Rusca dam

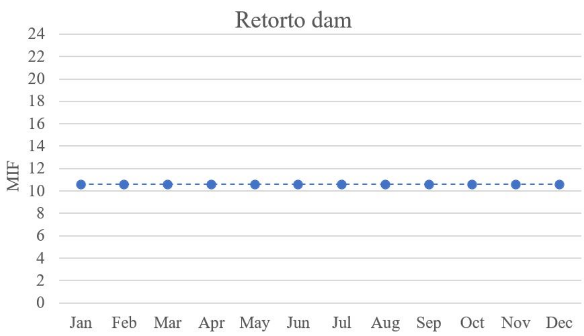


Figure 3.15: MIF Retorto dam

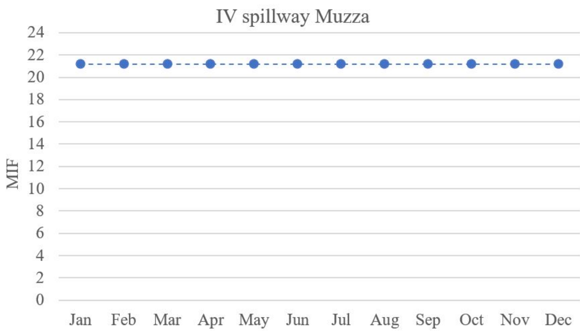


Figure 3.16: MIF IV spillway Muzza

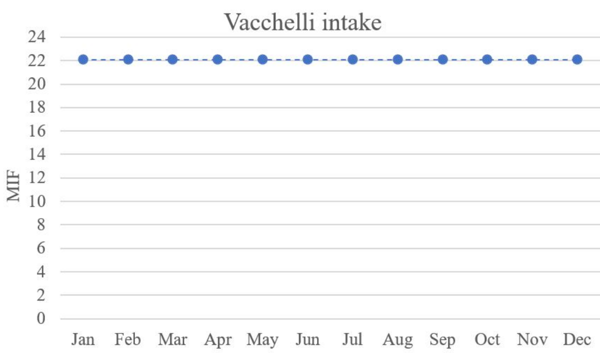


Figure 3.17: MIF Vacchelli intake

The presence of many withdrawals and hydropower plans provides lack of river connectivity, especially for fish fauna. Potential for passage of fish fauna is a critical point, and needs to be ensured to mitigate the impact of water diversion, and local change of morphology give nby the barrage. FAO guidelines(Fish Passes Design, Dimensions and Monitoring, Roma, 2002), gives the information for the design of fish passages, for both low impact (natural looking, made by boulders and stones such as rockfill ramp) and technical passages, such as pool pass.

A minimum runoff of $0.5m^3/s$ is always necessary to guarantee the functioning of these devices. Along Adda there are many MIF hydropower plants, turbining also the minimum instream flow, but also these plants have to leave the $0.5m^3/s$ for the fish passages. For the case of Muzza node, the fish pass is installed in order to control also the MIF, here the minimum runoff has to be at least $4m^3/s$.

Figure 3.18: Muzza fish pass (Low impact passage)

Figure 3.19: Sant'Anna dam, fish pool pass (Technical passage)

3.5 Downstream stakeholders

The are many stakeholder involved along river Adda, downstream Olginate's dam water is allocated based on concessions given to Consorzio's users, that finance the regulation and management work made by Consorzio itself.

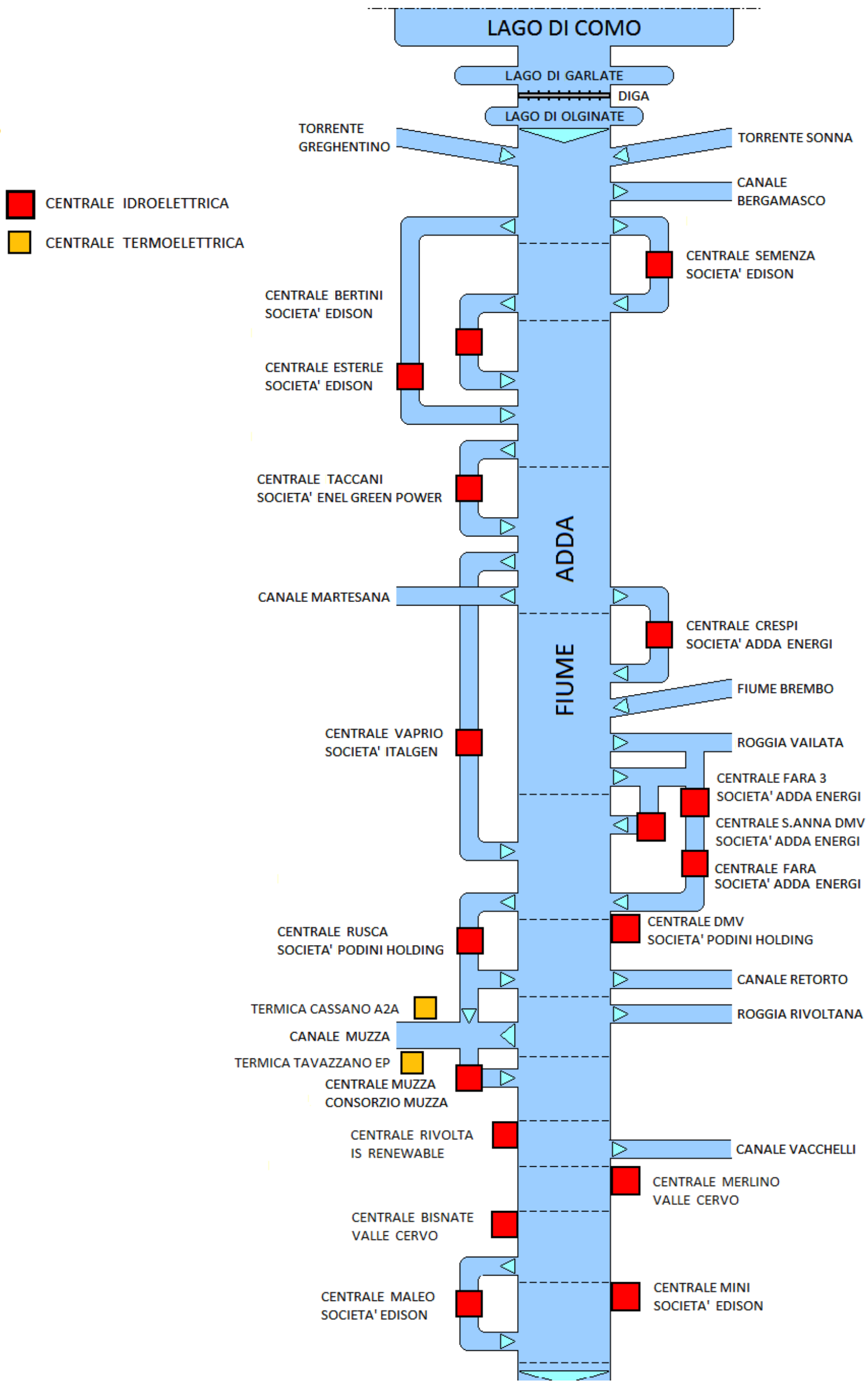


Figure 3.20: Withdrawals and hydropower plants scheme [Di Patti et al., 2013]

There are 18 hydropower plants in total (red squares in Fig. 3.20), 9 of them are MIF plants (turbining the MIF, but always leaving at least $0.5 m^3/s$ for the fish pass). There are also 7 irrigation channels and 4 tributaries: creek Greghentino, creek Sonna, river Brembo and river Serio.

3.5.1 Hydropower plants

All the 18 hydropower plants (H.P.) are run-of-river power plants, so they don't storage or waste water, but return all the amount of water to the river, at least with a delay. For the mass balance along the river they can be considered as irrelevant, their importance is given by their installed capacity that is about 106 MW, with a maximum daily production of 2.5 GWh.
It was taken a performance coefficient for the turbines of 0.85, to be on safe side.

Table 3.5: Hydropower plants along river Adda

Hydropower Plant	Energy Company	Q_{max} $[m^3/s]$	ΔH $[m]$	P_{max} $[kW]$	E_{max} $[kWh/day]$
Esterle H.P.	Edison	80.0	38.8	25878.8	621091.6
Semenza MIF H.P.	Edison	70.0	9.1	3034.0	72815.4
Bertini H.P.	Edison	51.0	29.0	12331.9	295964.7
Taccani H.P.	Enel GreenPower	180.0	7.8	11702.5	280859.5
Vaprio H.P.	Italgen	134.6	16.8	18793.3	451039.1
Crespi H.P.	Adda Energi	60.0	4.8	2415.5	57972.3
Fara MIF H.P.	Adda Energi	25.0	6.5	1354.5	32506.9
FaraVecchia H.P.	Adda Energi	65.0	4.9	2627.6	63063.4
Fara3 H.P.	Adda Energi	42.0	1.9	664.3	15944.4
Rusca MIF H.P.	Podini Holding	20.4	4.4	739.7	17751.8
Rusca-Retorto H.P.	Podini Holding	21.0	6.0	1050.2	25205.3
Rusca-Muzza H.P.	Podini Holding	135.0	8.0	9001.9	216045.8
Muzza MIF H.P.	Consorzio Muzza	80.0	4.7	3160.7	75856.1
Rivolta MIF H.P.	IS Renewable	50.0	3.8	1562.8	37508.0
Merlino MIF H.P.	Valle Cervo	80.0	4.6	3067.3	73615.6
Spino MIF H.P.	Valle Cervo	34.2	4.0	1140.2	27365.8
Pizzighettone old MIF H.P.	Edison	100.0	4.2	3500.7	84017.8
Pizzighettone new MIF H.P.	Edison	120.0	4.2	4200.9	100821.4
TOT		1348.2	163.4	106226.9	2549444.9

$$P_{max} = \rho_w g Q_{max} \Delta H \quad [kW] \tag{3.1}$$

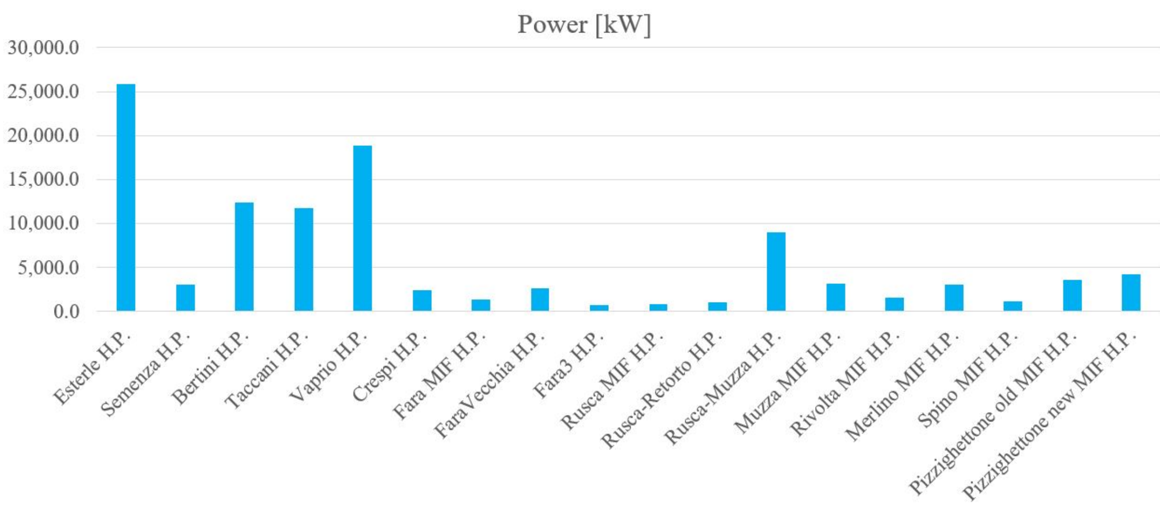


Figure 3.21: Hydropower plants power production

3.5.2 IRRIGATION CHANNELS

There are 7 irrigation channels withdrawing water from river Adda, in total the irrigation network absorbs $230 m^3/s$ in the trimester of major consumption, feeding a total area of $131400 ha$, entirely located in Lombardy.

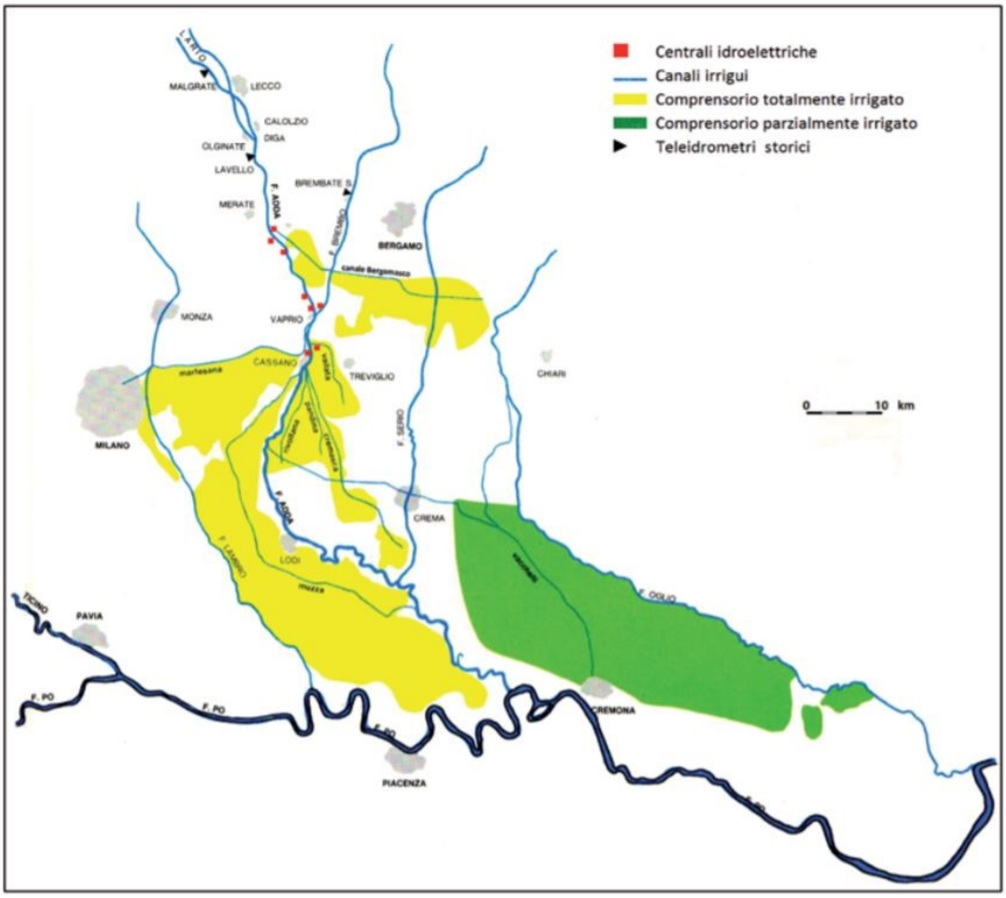


Figure 3.22: Layout of irrigation district [Di Patti et al., 2013]

In Fig. 3.22 in red the hydropower plants, in light blue the irrigation channels and tributaries, in yellow the totally irrigated area and in green the partially irrigated area.

Water allocation is based on concession contracts between Consorzio dell'Adda and the other consortium members (www.addaconsorzio.it):

- Consorzio di Bonifica Media Pianura Bergamasca
- Consorzio di Bonifica Est Ticino Villoresi
- Consorzio Roggia Vailata c/o Comune di Cavelzano
- Consorzio del Canale Retorto c/o Consorzio Miglioramento Fond. II Grado
- Comune di Rivolta d'Adda
- Consorzio di Bonifica Muzza-Bassa Lodigiana
- Consorzio Irrigazioni Cremonesi

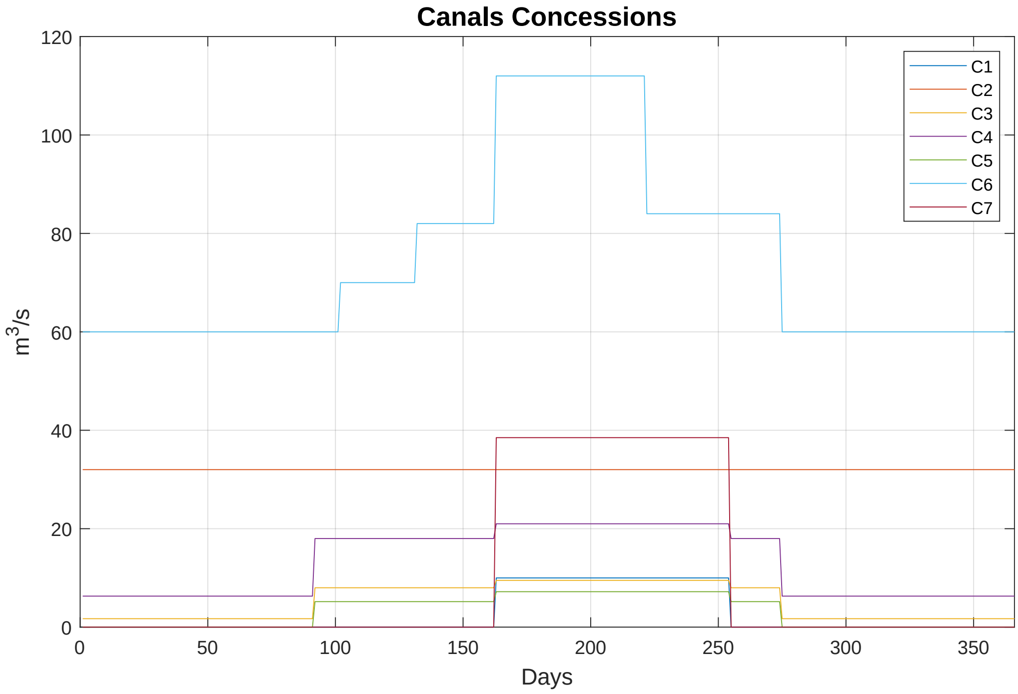


Figure 3.23: Water concessions for irrigation channels (Acque Nuove 2018, Consorzio dell'Adda)

Table 3.6: Water concessions for irrigation channels (Acque Nuove 2018, Consorzio dell'Adda)

Season	Winter	Summer	Summer- Acque Nuove
Period	01/10-31/03	01/04-30/09	11/06-10/09
C. Bergamasco C1	-	-	10.0
C. Martesana C2	32.0		32.0
D. Vailata C3	1.715	8.0	9.5
C. Retorto C4	6.3	18.0	21.0
D. Rivoltana C5	-	5.18	7.2
C. Vacchelli C7	-	37.0	38.5

	Winter	Summer				Summer - Acque Nuove	
	01/10-10/04	11/04-10/06	11/05-10/06	11/06-20/08	21/08-30/09	11/06-20/08	21/08-30/09
C. Muzza C6	60.0	70.0	82.0	110.0	82.0	112.0	84.0

As can be noticed from the graphic canal Muzza is the most important channel withdrawing quantity of water greater than the sum of all the other channels.

Actually canal Retorto is subdivided into two smaller channels: C. Cremasca and C. Pandina, the water is divided among them following the general rule of 3/5 of water for C. Cremasca and the remaining 2/5 for C. Pandina, behind what happens in reality there are other little agreements within their consortium, not explained in this work.

During summer period irrigation channels water supply is a priority for Consorzio dell'Adda, indeed hydropower plants have water concession over river runoff and can exploit that amount of water for power production. Irrigation users make a deal for water concession with Italian Government , the management authority, that is Consorzio dell'Adda, has the task to provide the established amount of water to each user, according to water availability in Adda river.

CHAPTER 4

MODELING OF ADDA RIVER SYSTEM

The path of river Adda is long and crosses territories of various nature, even for an approximated model of river basin it's necessary to well understand where there are entries or withdrawals and also possible water losses. Starting from Fig. 3.20 is done a scheme of stakeholders and their location. River Adda is subdivided into many segments, upstream and downstream limited by control cross sections, and each one of them is solved by continuity equation. The modeling of Adda makes possible to simulate different policies for water allocation starting from the flow rate delivered by Olginate's dam, showing a plausible power production for all the system depending on water management decisions applied.

Figure 4.1: River Adda downstream Lake Como, satellite image

4.1 Understanding of the region and satellite investigations

A good model reflects as much as possible reality, Fig. 4.2 shows the principal nodes interesting the river path. Some intersections are located in the same point, for the application of continuity equation in each segment it's fundamental to know precisely which flow derivation is before, for this reason deeper satellite investigations are made.

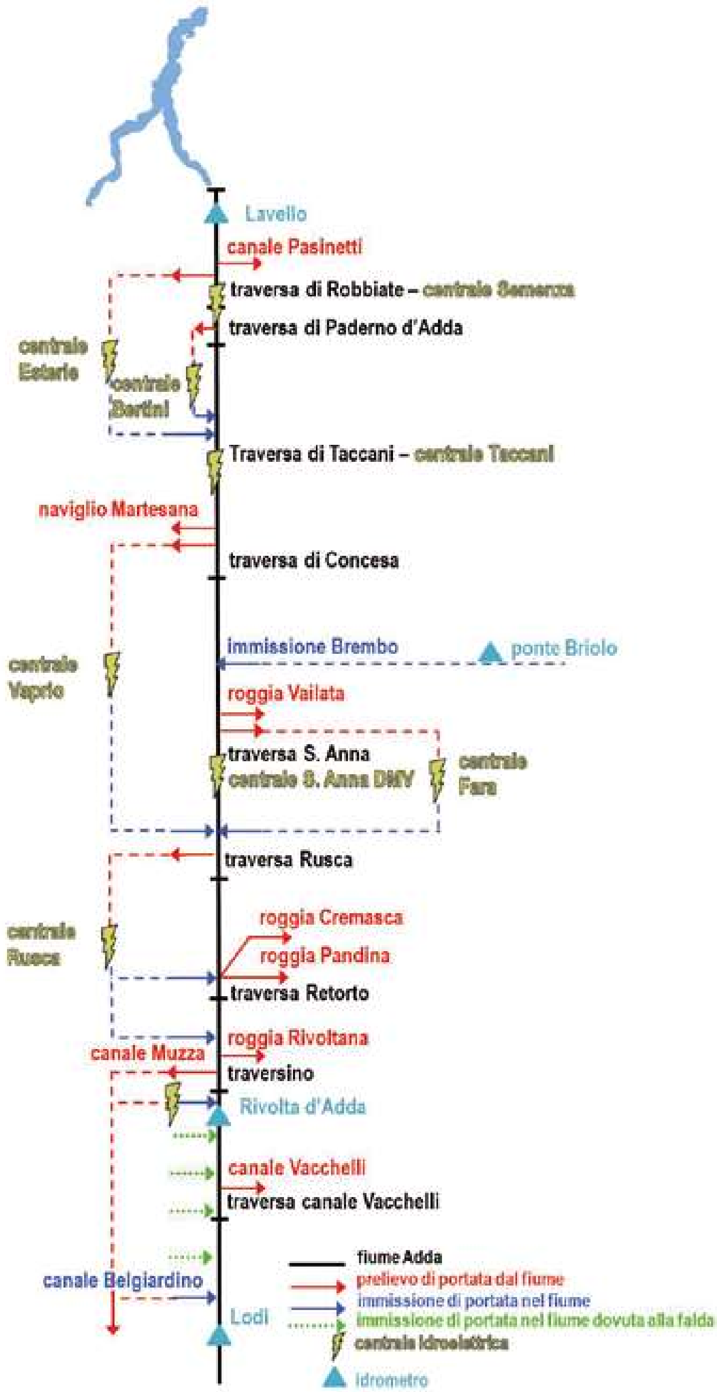


Figure 4.2: River Adda downstream Lake Como, hydraulic scheme till Lodi [ConsorzioDell'Adda, 2014]

Figure 4.3: Sant'Anna Dam and Fara hydropower plants

Fara's node functioning is quite complex to understand. The three hydropower plants are owned by Adda Energy, so the real administration of water distribution is internal to the company. Fara MIF H.P. is the plant with the highest jump, it can also turbine water of MIF, always leaving $0.5m^3/s$ for fish passage; for this reason generally flow rate is left only to this plant till it runs at full capacity and then the water in excess is delivered to the other two.

The water inputs from Vaprio H.P. and Fara 3 H.P. are unclear, since it's an addition of water flow the MIF is surely satisfied, even if is a bit inaccurate, it's assumed that upstream there is the inflow coming from Vaprio H.P. and then from Fara 3 H.P.

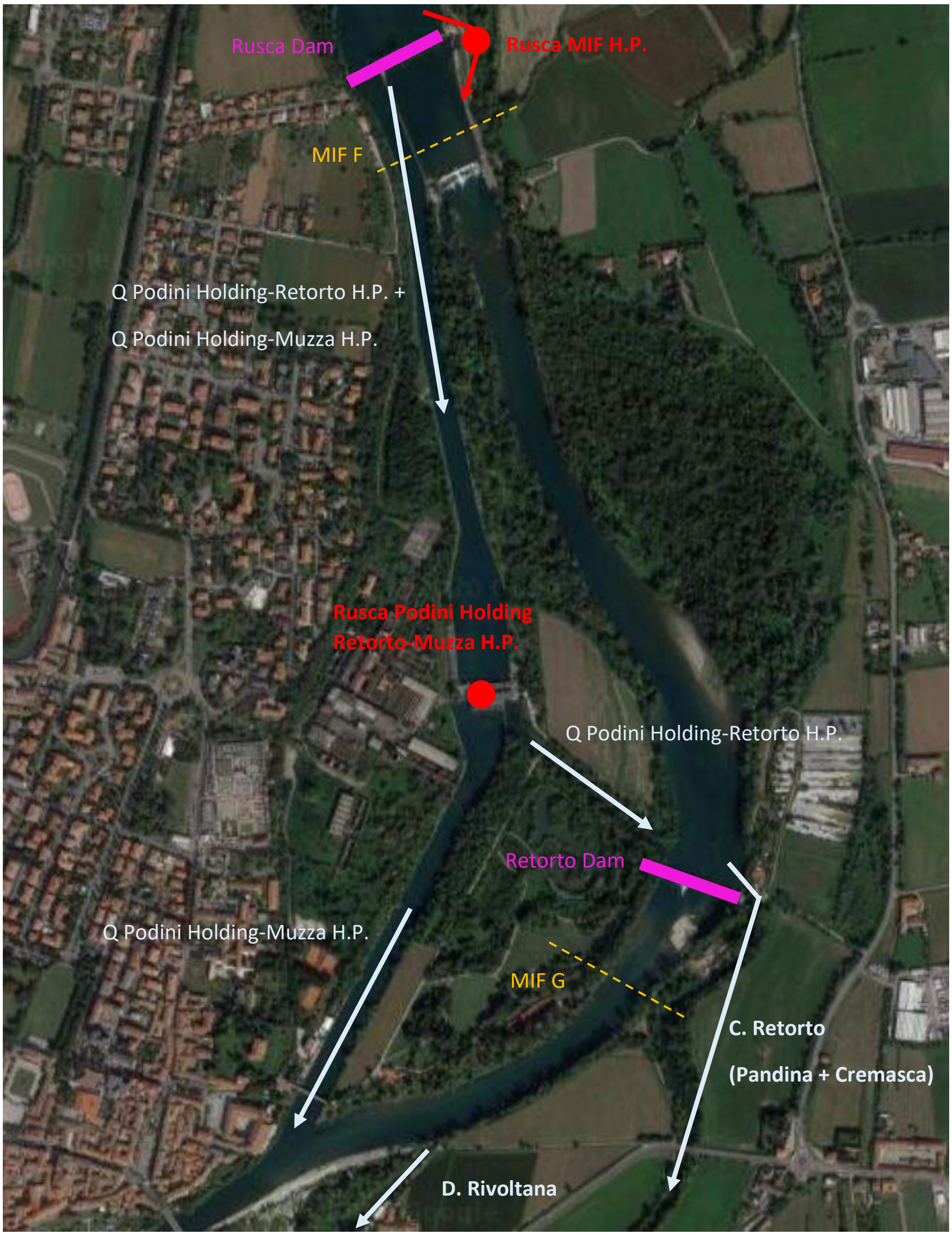


Figure 4.4: Rusca Dam, Retorto Dam and Rusca H.P.

Podini Holding owes two hydropower plants in Cassano d'Adda; the biggest one derives water from a diversion channel starting from Rusca Dam, this plant can turbine water through two different systems of turbines, one linked with the jump in direction of channel Retorto, the other in towards channel Muzza (this last has a greater capacity).

During summer period, channel Retorto has priority compared to hydropower plants, for this reason P.H. Company usually prefers to turbine water with the jump in Retorto direction, so as to return water before canal Retorto intake.

Figure 4.5: Canal Muzza Node 1

In Muzza's node are present many spillways, working when there is an excess of water flow, in spillway number three there is an hydropower station to exploit this amount of water. San Bernardino Dam guarantees to the thermoelectic plant a flow rate of $10m^3/s$, necessary for cooling machines. The management of spillways is done by Consorzio della Muzza, always respecting MIF impositions.

There are three MIF cross sections in Muzza's node, each one of them located after a spillway discharge. Since we want to reduce channel Muzza's diversion to an unique segment and since spillways activation is a task interesting only of Consorzio della Muzza, the other MIF are neglected and the most downstream MIF (that is also the biggest) is considered the MIF at the end of the node, as approximation choice.

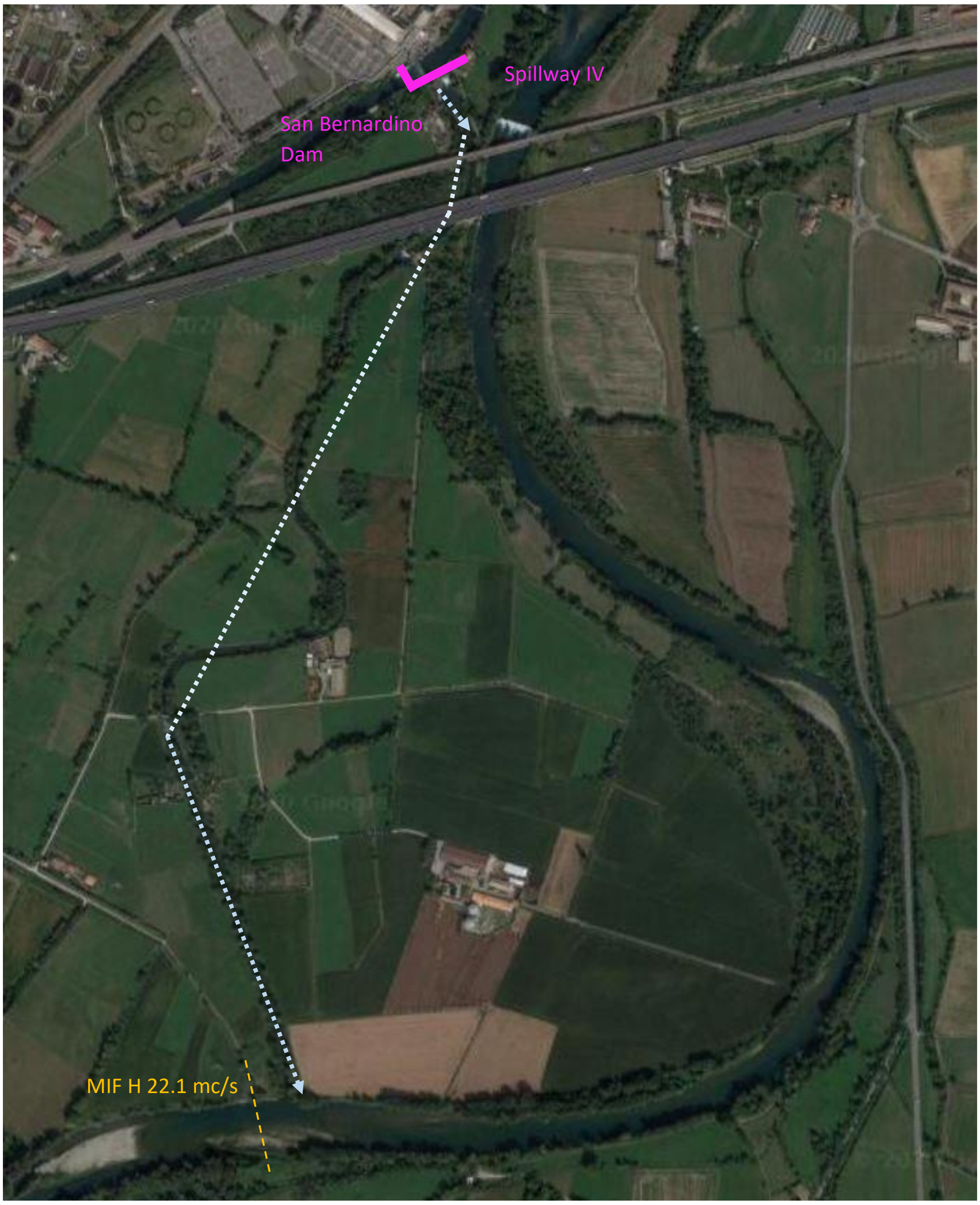


Figure 4.6: Canal Muzza Node 2

4.2 Tributaries and flow rate contributions

Along river Adda there are 4 main tributaries: creek Greghentino, creek Sonna, river Brembo and river Serio; their entrance location is showed in scheme in Fig. 3.20, c. Ghreghentino and c. Sonna input is immediately after the lake of Olginate, r. Brembo falls into river Adda downstream Crespi d'Adda and r. Serio upstream Pizzighettone.

The first two creeks: Greghentino and Sonna are of low discharge, around $1 m^3/s$, their regime is highly variable with precipitations and often influenced by groundwater flow coming from the big lakes upstream. Since the two are located before any withdrawal and their flow rate is quite small compared to the quantity of water delivered by the dam, their contribution is neglected for the approximated model of Adda.

River Serio has a mean flow discharge about $10-20m^3/s$. It is an important tributary but the fact that its entrance is placed downstream all the withdrawals and upstream only the two hydropower plants of Pizzighettone, makes r. Serio meaningless for water allocation. It is taken a symbolic runoff of $10m^3/s$, that will be used only for the simulation of power production in Pizzighettone H.Ps.

River Brembo's entrance is located after Crespi d'Adda, in the first half of Adda's path, knowing its discharge is very important for downstream withdrawals.

The data series given report Brembo's discharges measured at Ponte San Pietro, after this point there are ditch Trevigliesi's intakes (ditch Moschetta and ditch Vignola), so to know the amount of water falling in Adda it's required to subtract this amount. Sometimes the result is negative, that is because of some errors in measurements, when it happens the flow discharge is set to zero.

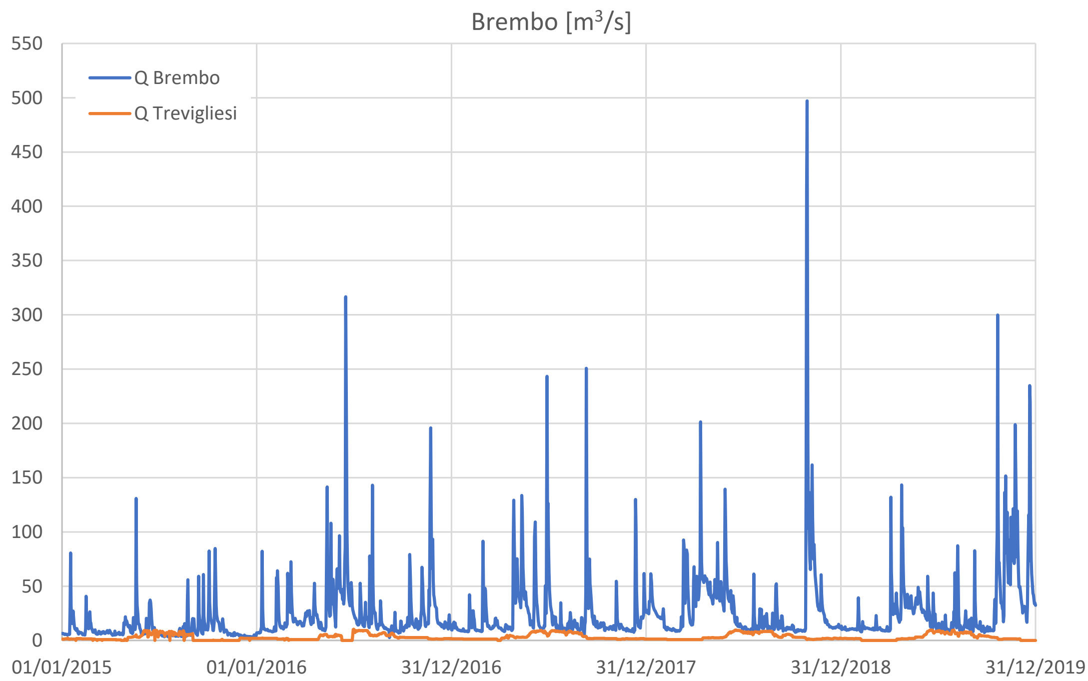


Figure 4.7: River Brembo and ditches Trevigliesi, years 2015-2019

To properly understand river Brembo flow discharge, falling in Adda, are made the Flow Duration Curves in summer period for each year analyzed. The FDC describes the probability of exceedance of a certain runoff value during the interval of time considered.

The mean quantile with a probability of exceedance of 50% is $9.11m^3/s$,

The mean quantile with a probability of exceedance of 75% is $4.08m^3/s$,

The mean quantile with a probability of exceedance of 80% is $3.56m^3/s$,

As approximation choice, in summer period it's taken a value of $4m^3/s$ for Brembo's discharge in the model.

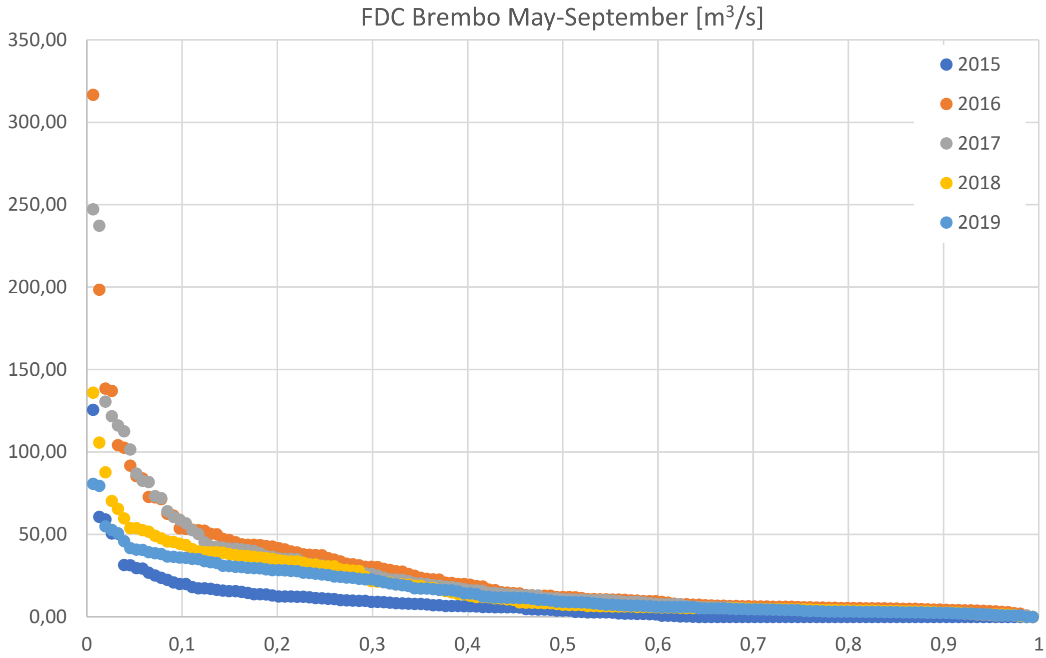


Figure 4.8: River Brembo Flow Duration Curves, years 2015-2019 summer period

We have to consider also hidden and not properly measured contributions along the river to correctly asset the hydraulic balance. In the second half of Adda path, starting from Rivolta d'Adda till Pizzighettone, the main feature of the area are the springs, which contributes to Adda runoff with a certain amount of water. The evaluation of a precise amount of water deriving from groundwater flow is complex and linked to many variable environmental factors regarding not only the area of interest but a great part of Lombardia region.
There are also other flow rate increase coming from irrigation drains of upstream areas and irrigation channels discharges.

During the years Consorzio dell'Adda and other collaborators have done many investigations and studies about this amount of water not registered. All the study was done in an empirical way: it was computed the difference of flow rate between the hydrometers of Rivolta d'Adda and Lodi (in Fig. 4.2), since there isn't any tributary in this stretch of river, the additional flow rate $\Delta Q[m^3/s]$ was converted into an inflow for unit of river length $[1/km]$:

$$q = \frac{\Delta Q}{L_{Rivolta-Lodi}} \quad [m^3/s * km] \tag{4.1}$$

With:
- $\Delta Q[m^3/s]$: increase of flow rate from Rivolta d'Adda till Lodi
- $L_{Rivolta-Lodi}[km]$: length of the stretch of River between Rivolta d'Adda and Lodi

In the same was also computed the additional flow rate from Lodi till Pizzighettone. In total not registered contributions in the second half of river Adda are assumed to be about $30m^3/s$, even if they vary during seasons.

They should be done deeper hydrological and hydraulic studies over the whole region to create a more precise model, reduce uncertainties and to properly asses these contributions; anyway this is not a priority since in the last part of the river path there is only one withdrawal, channel Vacchelli, which exploits the water available.

4.3 River Adda approximated simulator

Adda simulator is a sequence one after the other of boxes, each one of them representing an hydraulic segment of river Adda and limited by control cross-sections. Any segment presents an external outflow or inflow and it's solved by continuity equation (mass balance) [Citrini and Noseda, 1987].

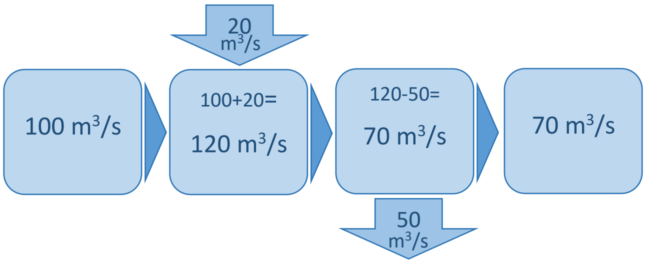


Figure 4.9: Scheme of the boxes representing approximated segments of river Adda

The tributaries, or inflows, are associated with a constant value as explained in Subsection 4.2; the irrigation channels, or withdrawals, are managed following the policy pre-established for water allocation and the intakes of hydropower plants are assumed to work always exploiting the maximum quantity of water available, according to the installed capacity of the plant.
In any segment water derivation must be done respecting the Minimum Instream Flow, only in the case of MIF H.P. with fish passages the runoff can be left only at $0.5m^3/s$.

The simulator can be set up with many INPUT about the system characteristics:

- Hydropower plants characteristics: hydraulic head, maximum flow rate (concession flow rate), efficiency, hours of production, estimated monetary value for energy...
- Irrigation channels characteristics: maximum flow capacity
- Minimum Instream Flow: value assigned in each river cross section
- Tributaries: value assigned for each tributary

In each simulation can be put as INPUT:

- Water flow delivered by Olginate's Dam $Q_0 [m^3/s]$
- Downstream water allocation policy

ESTRELE H.P. - Edison	
Data:	
Q_{max} [m³/s]	80.00
ΔH [m]	38.81
$\eta_{efficiency}$	0.85
Installed capacity[kW]	25,879
Production hours [h]	24
Energy max producible/day [kWh/day]	621,092
Energy max producible/week [kWh/week]	4,347,642
Energy max producible/month [kWh/month]	18,632,749
Energy max producible/year [kWh/year]	226,698,450

General info:
Name: Centrale "Carlo Estrele"
Location: Cornate d'Adda (MI)
Type of plant: run-of-river power plant
Turbine: Francis

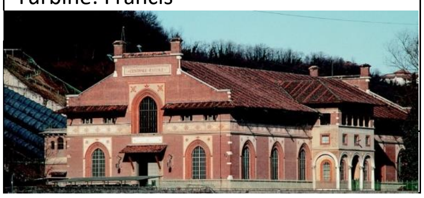


Figure 4.10: Estrele Hydropower Plant - Edison

Once the simulation is started, basing on the INPUT the amount of water coming from the dam is diffused in the system through the various branches.

The OUTPUT of the simulation are:

- Effective water flow derived in each irrigation channel
- Runoff in each section of the river
- Hydroelectic power generation, of the single power plant and of the whole system

The hydropower production is computed and compared with the maximum installed capacity. The energy production is computed too, the percentages resulting from the comparison with the maximum energy producible will be the same of power generation, unless the production hours are different from 24h; the colors legend shows immediately the relationship between production in course and maximum generating potential (once assigned simulator settings). The simulator can also estimate possible earnings deriving from power production, once set the energy price [$\$/kWh$].

Production simulation:	
Q [m³/s]	77.70
Power production [kW]	25,135
Production hours [h]	24
Energy produced /day [kWh/day]	603,235
Energy produced /week [kWh/week]	4,222,647
Energy produced /month [kWh/month]	18,097,058
Energy produced /year [kWh/year]	220,180,869

Figure 4.11: Example: Estrele Hydropower Plant - Edison. Production simulation

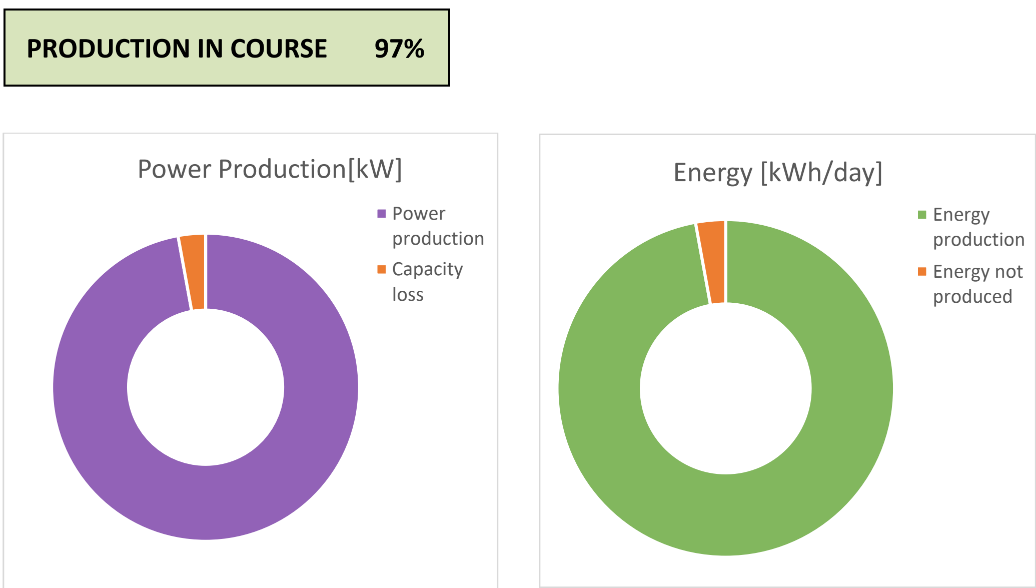
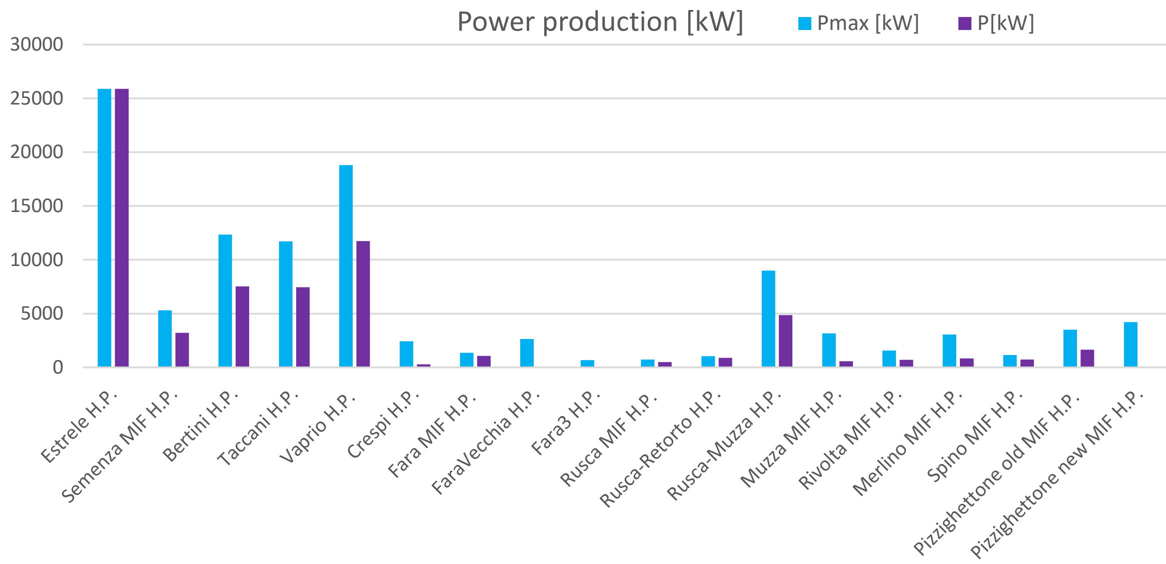


Figure 4.12: Example: Estrele Hydropower Plant - Edison. Production in course



Summary of the power production in course:

	Q [m³/s]	ΔH [m]	P[kW]	E [kWh/day]	% max
Estrele H.P.	80.0	38.8	25,878.8	621,091.6	100%
Semenza MIF H.P.	42.5	9.1	3,223.6	77,366.4	61%
Bertini H.P.	31.1	29.0	7,520.0	180,480.5	61%
Taccani H.P.	114.6	7.8	7,450.6	178,813.9	64%
Vaprio H.P.	84.0	16.8	11,727.5	281,459.7	62%
Crespi H.P.	7.0	4.8	281.8	6,763.4	12%
Fara MIF H.P.	19.8	6.5	1,072.7	25,745.5	79%
FaraVecchia H.P.	0.0	4.9	0.0	0.0	0%
Fara3 H.P.	0.0	1.9	0.0	0.0	0%
Rusca MIF H.P.	13.3	4.4	482.2	11,573.5	65%
Rusca-Retorto H.P.	17.5	6.0	875.2	21,004.5	83%
Rusca-Muzza H.P.	73.0	8.0	4,867.7	116,824.8	54%
Muzza MIF H.P.	14.3	4.7	565.0	13,559.3	18%
Rivolta MIF H.P.	22.1	3.8	690.3	16,568.0	44%
Merlino MIF H.P.	21.6	4.6	828.2	19,876.2	27%
Spino MIF H.P.	21.6	4.0	720.2	17,283.7	63%
Pizzighettone old MIF H.P.	46.6	4.2	1,631.3	39,152.3	47%
Pizzighettone new MIF H.P.	0.0	4.2	0.0	0.0	0%
TOT	**609.0**	**163.4**	**67,815.1**	**1,627,563.1**	**63%**

P	E	%prod. max
108,502	2,604,056.4 kWh	100
81,377	1,953,042.3 kWh	75
54,251	1,302,028.2 kWh	50
27,126	651,014.1 kWh	25

Colors legend for power production:	
(red)	0% Emax
(orange)	<25% Emax
(yellow)	25%≤ <50% Emax
(light green)	50%≤ <75% Emax
(green)	75%≤ <100% Emax
(blue)	100% Emax

Figure 4.13: Example: River Adda system hydropower production. Power generation control panel

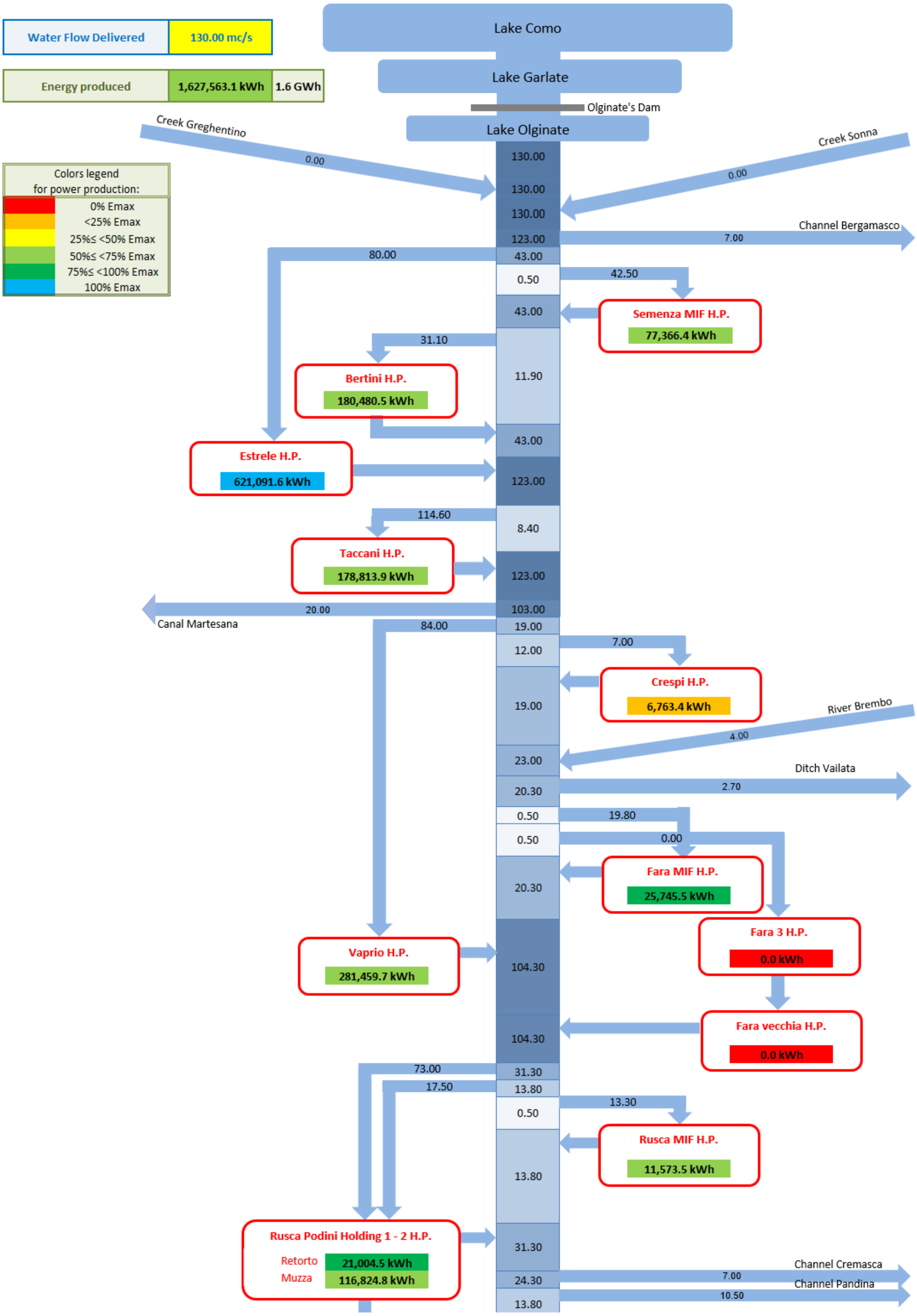


Figure 4.14: Example: River Adda approximated simulator. Interface I part

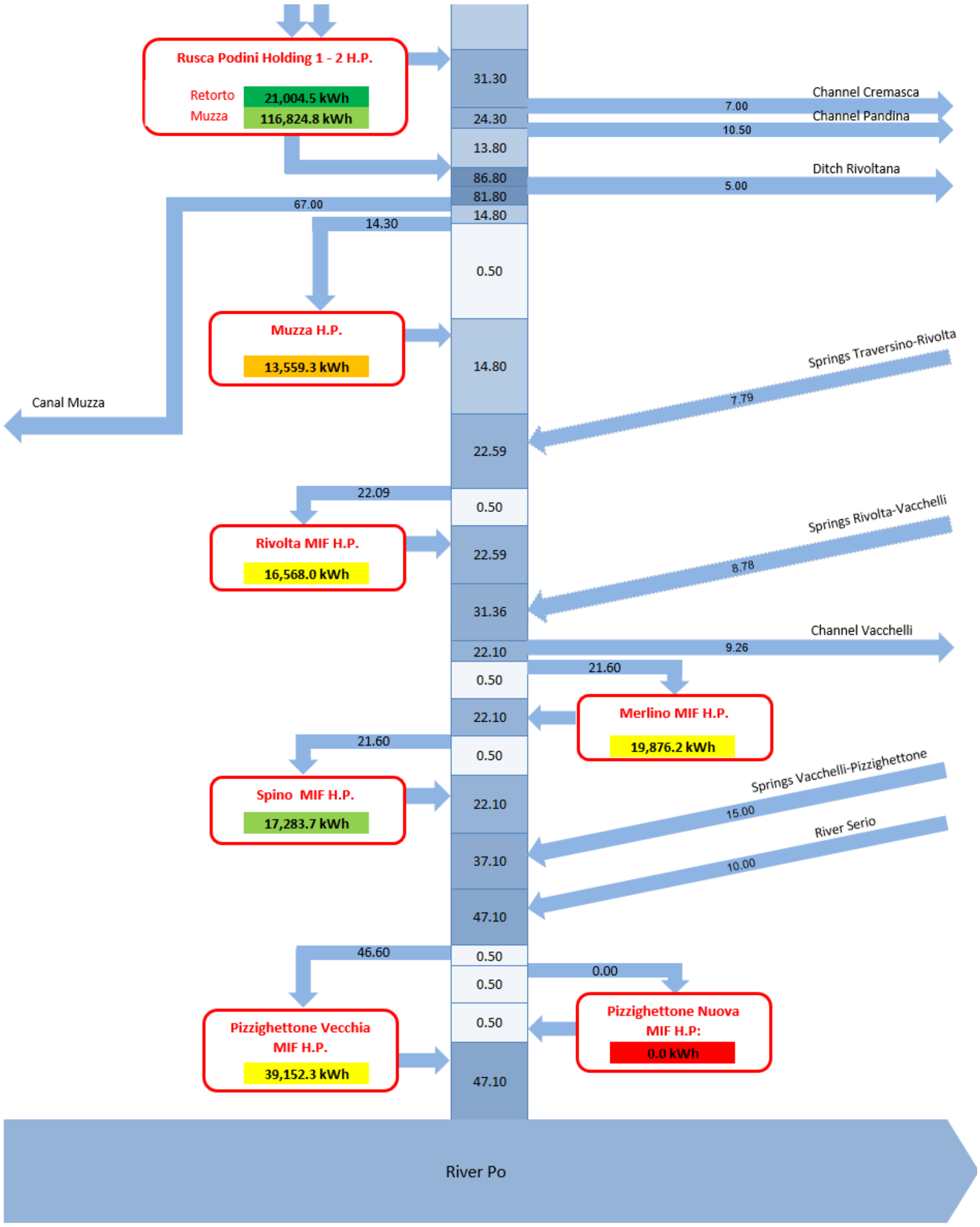


Figure 4.15: Example: River Adda approximated simulator. Interface II part

Chapter 5

Downstream water allocation

It is a challenging and important issue to make efficient water sources management and water allocation strategies that optimize economic and social well-beings in a fair and equitable manner without conceding the sustainability of ecosystems. Nevertheless, water allocation between competing demands could be made administratively subject to important objectives, such as effective resources utilization, equity and sustainable environment [Dinar, A. Rosengrand, M. and Meizen-Dick, 1997]. Another difficulty associated with water resources management comes from the fact that many water using activities generate externalities downstream. Therefore, significant impacts of slight adjustment in agricultural water allocation on economic and hydrological well-fares will occur [Chang and Wang, 2013]. Since water flows downstream together with externalities, the natural spatial scale at which water allocation decisions can be made is the river basin. Policy instruments designed to achieve a certain level of economic efficiency and equity should therefore best be developed and implemented at that scale, from the top till the bottom, involving the whole system. When crop irrigation is involved, water is usually allocated by a system of annual rights to use a fixed, static volume of water, which is typically less than what farmers would expect. Farmers' demand for water is derived from the value of its use in crop production, which in turn depends on crop water requirements and crop prices. Non-consumptive water rights held by power companies cannot negatively affect prior consumptive water rights enjoyed by farmers, hydropower companies are bound to respect pre-defined monthly release targets reflecting agricultural demands, for this reason in most of the cases agriculture has priority for water allocation [Tilmant et al., 2009].

Figure 5.1: Stakeholders involved in river basin management [Loucks and Van Beek, 2005]

5.1 Is there enough water to satisfy water demand?

Answering this question is the first step for solving water allocation problem. For the purpose of evaluating if in effective there is enough water to satisfy all the water demand, it's done a comparison between the cumulative volumes of the canals concessions and the ones from inflows. The water concessions for irrigation channels, as shown in Tab.3.6, have the highest values in summer period, when there is the maximum exploitation of water for agricultural production.
In Fig.5.2 are represented the cumulative volumes of canals concessions, it can be noticed that at the beginning of summer period, on day 11/06 corresponding to day 163 in the graphic, there is a huge hike of the water demand. The blue line represents the sum of all the canals concessions, so the total water volume to be provided to users.

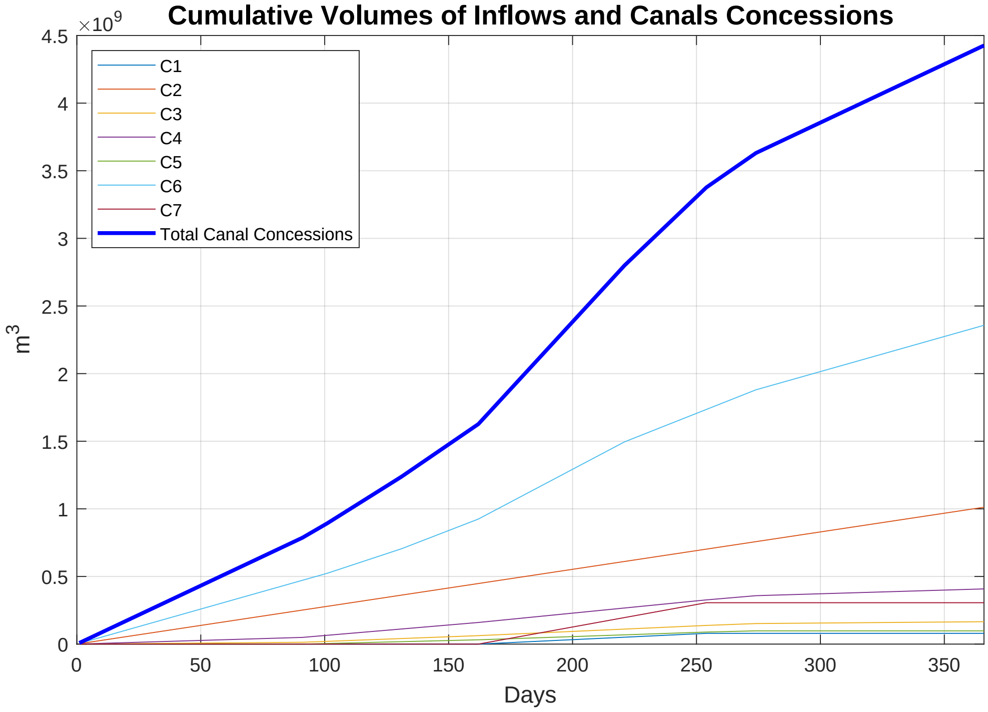


Figure 5.2: Cumulative volumes of canals concessions $[m^3]$

Withdrawals from rivers can be constrained or not, depending on the combination of the water needs, varying in space and time, that have to be satisfied. When the river runoff is greater than the MIF and the water demand, there is not a particular need to manage the resource, but only to verify that environmental constrains are fulfilled. In the other cases, the withdrawal is constrained and the water demand can't be completely satisfied [Loucks and Van Beek, 2005].

Once known the total amount of water needed during the year, it's done a comparison with the total amount of water available, so the water from inflows; this is done through the graphic of inflows cumulative volumes in the historical series.
In the case of study the river runoff can be approximated to the delivered outflow by Olginate's dam. As it was explained in Section 4.2, along the river path there are many flow rate contributions which vary during the year, at the same time must be always guaranteed the Minimum Instream Flow specific for any river section, MIF maximum value along the water course is approximately $20 m^3/s$. For most of the year MIF will be balanced and guaranteed by inflows coming from tributaries and contributions, in very drought periods instead MIF will be considered as a sort of "additional user" and its flow rate will be subtracted from the delivered outflow $Q_0 [m^3/s]$.

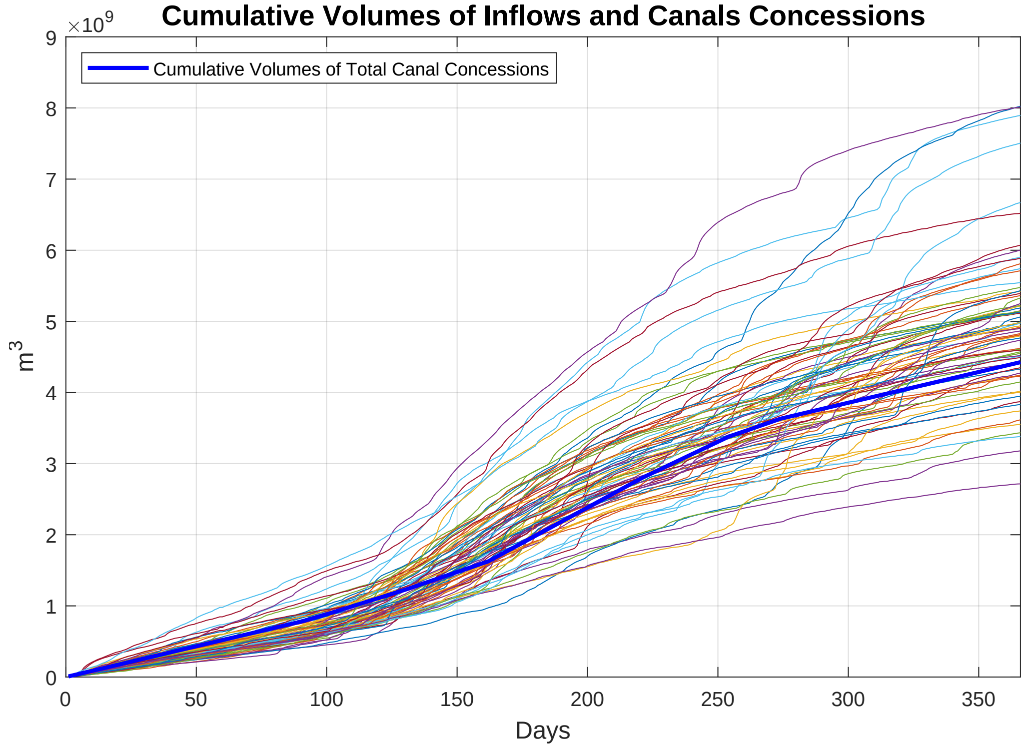


Figure 5.3: Cumulative volumes of inflows and canals concessions [m^3]

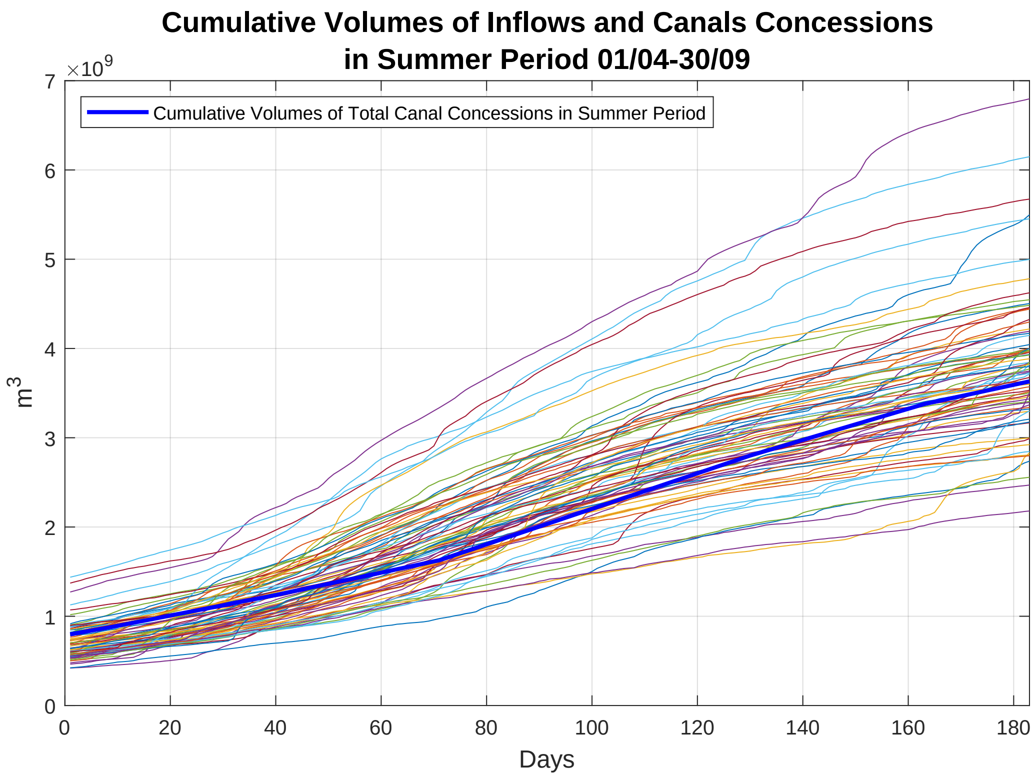


Figure 5.4: Cumulative volumes of inflows and canals concessions in summer period 01/04-30/09 [m^3]

At a first sight the "blue line" of cumulative volumes of total canals concessions stays more or less in the middle of the historical data, this means that for almost half of the series it isn't possible to fully satisfy the water demand. Starting from the beginning of June, the trend of canals concessions is steeper than the one of inflows, this requires a deeper analysis of the summer period.

The quantity of water above the "blue line" is the water in excess, that in one side can be stored in the reservoir, but on the other side it can be very dangerous if overflows the reservoir's capacity.

In early summer period, Fig. 5.5 there is a great water availability, that decreases more and more with the advance of the season. If it were possible to store all the water in excess, the amount of water would be sufficient for all the season, but in the case of Lake of Como the space it's very small as the water storable, staying on safe side.

For this reason the withdrawal is considered constrained, and hardly ever fully satisfied. The regulation policy will be based on the idea of "save water" to guarantee its availability for all the summer period, often fulfilling only in part the demand even when indeed there is water. This farsighted point of view is the only way to prevent what happens in Fig. 5.7 where the inflows alone are not enough to satisfy the users needs.

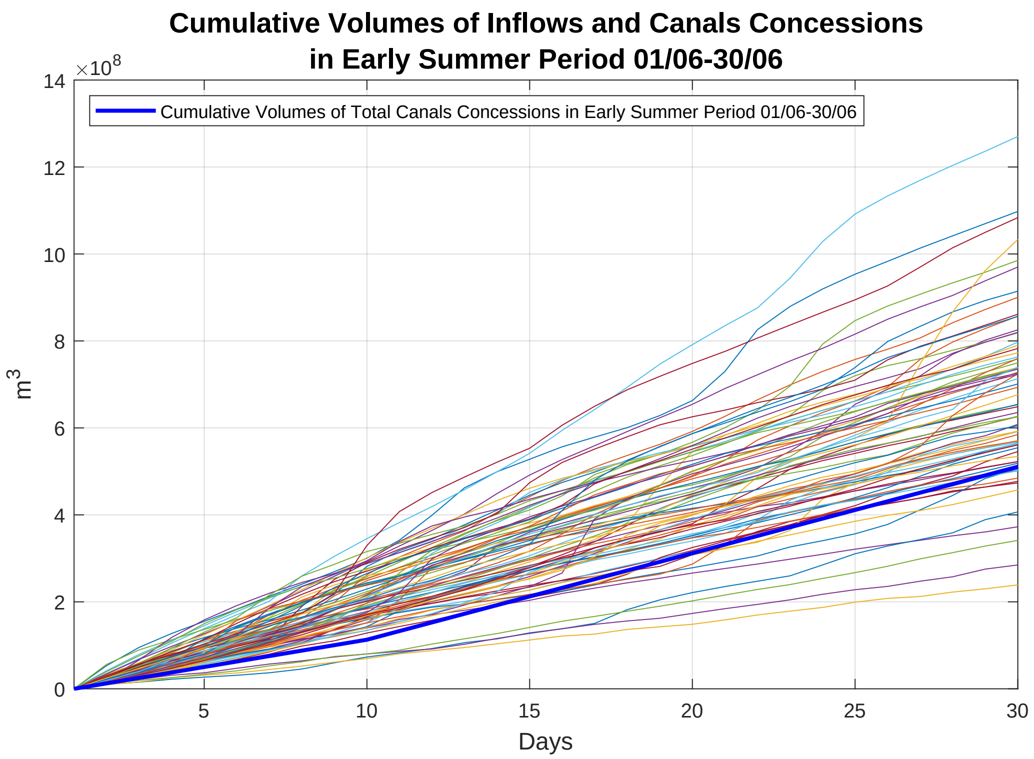


Figure 5.5: Cumulative volumes of inflows and canals concessions in early summer period 01/06-30/06 [m^3]

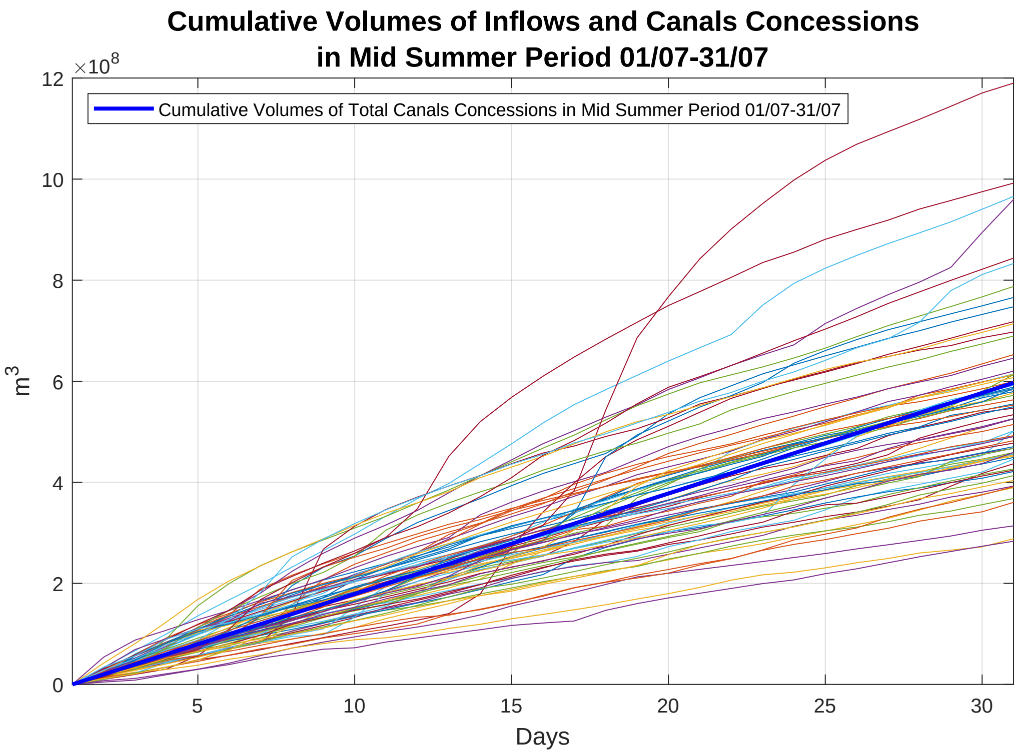


Figure 5.6: Cumulative volumes of inflows and canals concessions in mid summer period 01/07-31/07 $[m^3]$

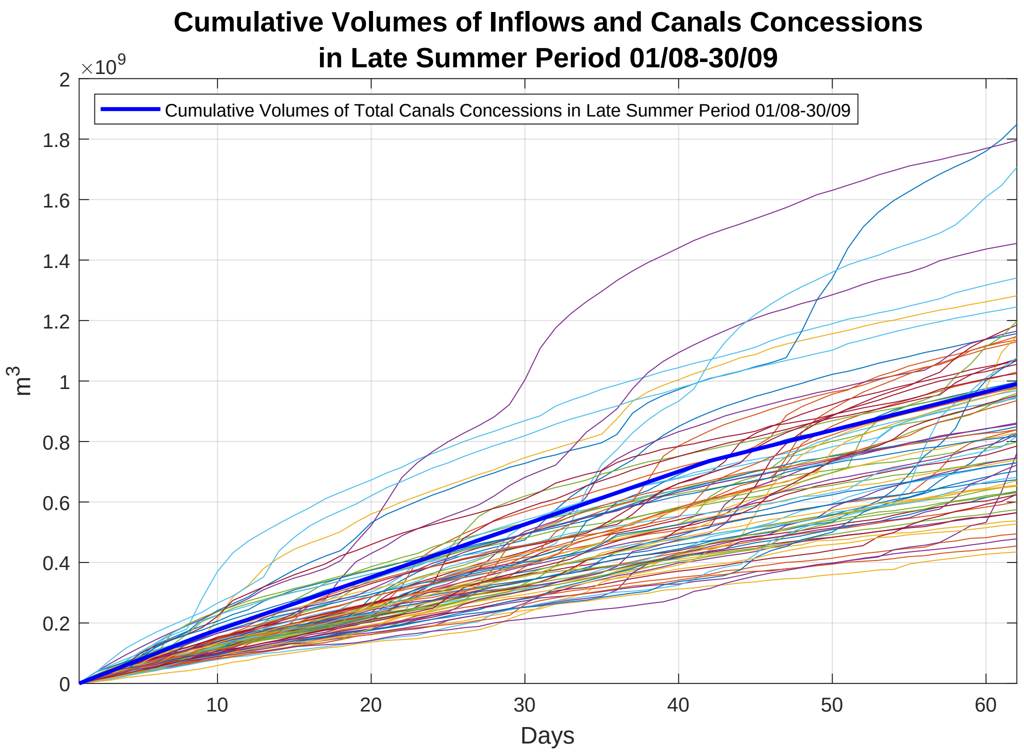


Figure 5.7: Cumulative volumes of inflows and canals concessions in late summer period 01/08-30/09 $[m^3]$

5.2 Analysis of Consorzio dell'Adda water allocation policy

Before researching an "optimized" water allocation policy, is analyzed the management strategy by Consorzio dell'Adda in the last 10 years. As explained before, in Tab.3.6, water concessions between channels are of different magnitude, this makes a direct comparison about the delivered flow rate in channels not very useful for our aim.

For the real comparison it's done a sort of adimensionalization: in each day for each channel it's taken the flow rate derived which is than divided for summer period water concession, the result is a percentage, the more the percentage is high the more the water derived is close to concession value.

In each day are summed up percentages coming from the 8 irrigation channels (considering c. Pandina and c. Cremasca separately), in a day during which in all the channels is derived a flow rate equal to the concession one, the sum of percentages will be 800%.

Below are reported some example years for the analysis, the graphics show that generally in summer period Consorzio dell'Adda's policy is to be proportional deriving water flow equally, especially when there is abundance of water; most of the time channel Bergamasco is kept with a constant flow, probable this choice is due to internal agreements. In the rest of the year, in particular in the first trimester, canal Muzza and Vacchelli are predominant, but we have to keep in mind that winter concessions are different and many channels are dry.

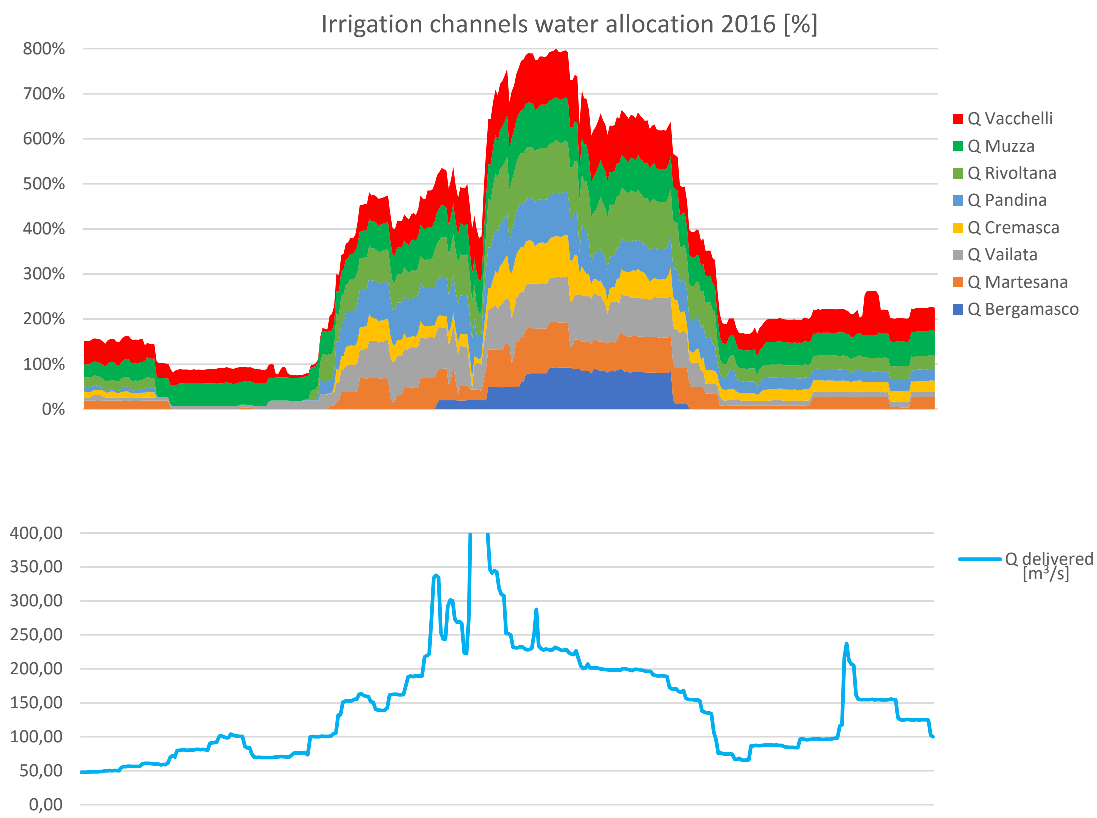


Figure 5.8: Irrigation channels water allocation 2016 [$\%Q_{conc}$]

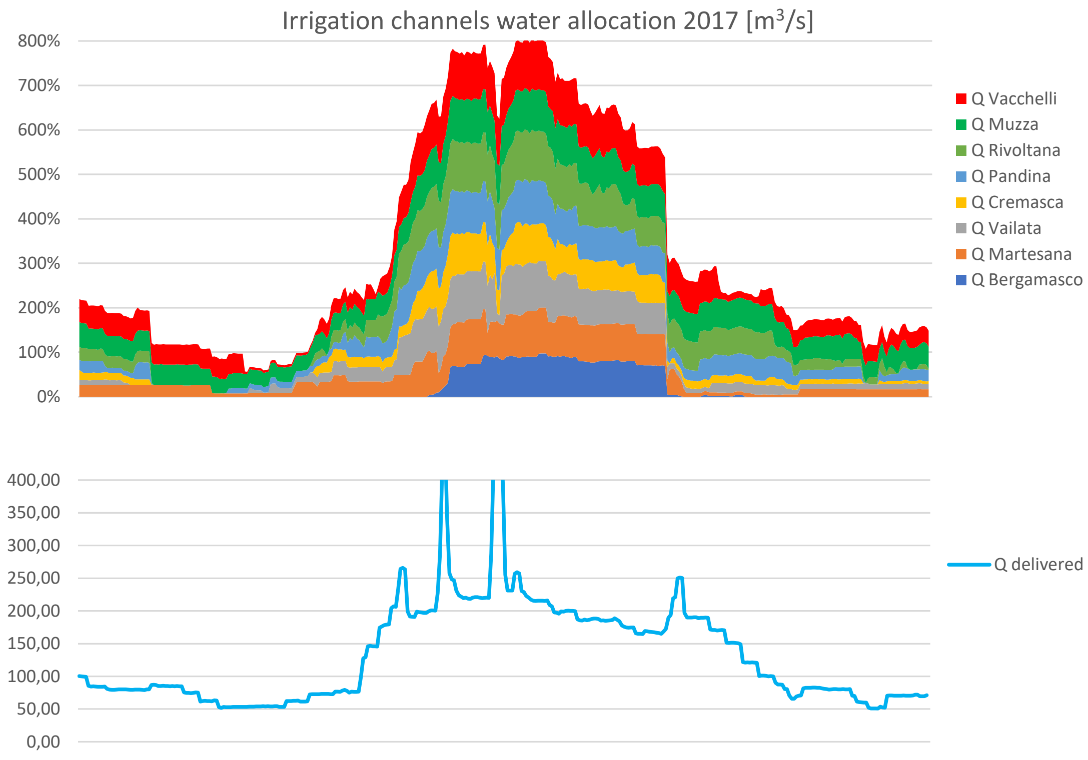


Figure 5.9: Irrigation channels water allocation 2017 [$\%Q_{conc}$]

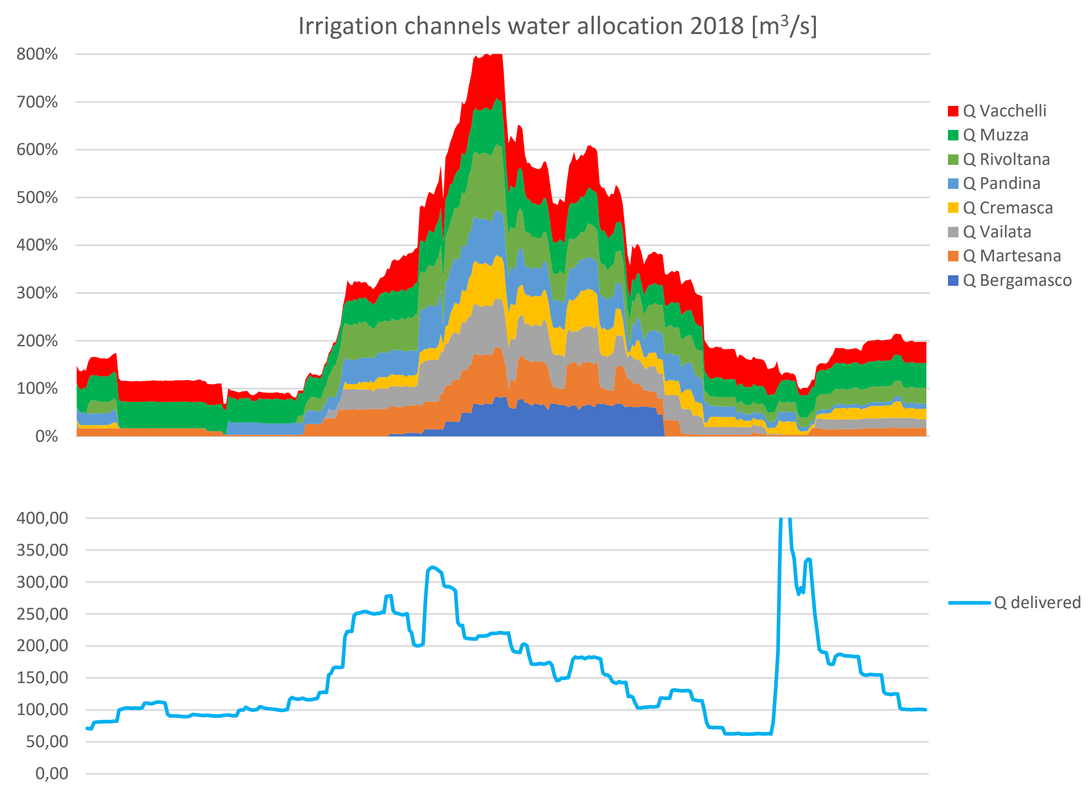


Figure 5.10: Irrigation channels water allocation 2018 [$\%Q_{conc}$]

5.3 Application of Dynamic Programming Network (DPN)

Dynamic programming is an optimization approach that transforms a complex problem into a sequence of simpler problems; its essential characteristic is the multistage nature of the optimization procedure. Dynamic programming is an approach that divides the original optimization problem, with all of its variables, into a set of smaller optimization problems, each of which needs to be solved before the overall optimum solution to the original problem can be identified. The water supply allocation problem, for example, needs to be solved for a range of water supplies available to each user. Once this is done the particular allocations that maximize the total net benefit can be determined.

A network of nodes and links can represent each discrete dynamic programming problem. Dynamic programming methods find the best way to get to any node in that network. The nodes represent possible discrete states that can exist and the links represent the decisions one could make to get from one state to another.

Thus, dynamic programming models involve states, stages and decisions. The relationships among states, stages and decisions are represented by networks, such as that shown in Fig. 5.11. The states of the system are the nodes and the values of the states are the numbers in the nodes. Each node value in this example is the quantity of water available to allocate to all remaining users, that is the quantity of water remaining in the river at a certain cross section. These state variable values typically represent some existing condition either before making, or after having made, a decision. The stages of the system are the separate columns of linked nodes. The links in this example represent possible allocation decisions for each of the users. Each stage is a separate user, in the problem analyzed there are 8 stage in total, as showed in Subsection 5.3.1. Each link connects two nodes, the left node value indicating the state of a system before a decision is made, and the right node value indicating the state of a system after a decision is made.

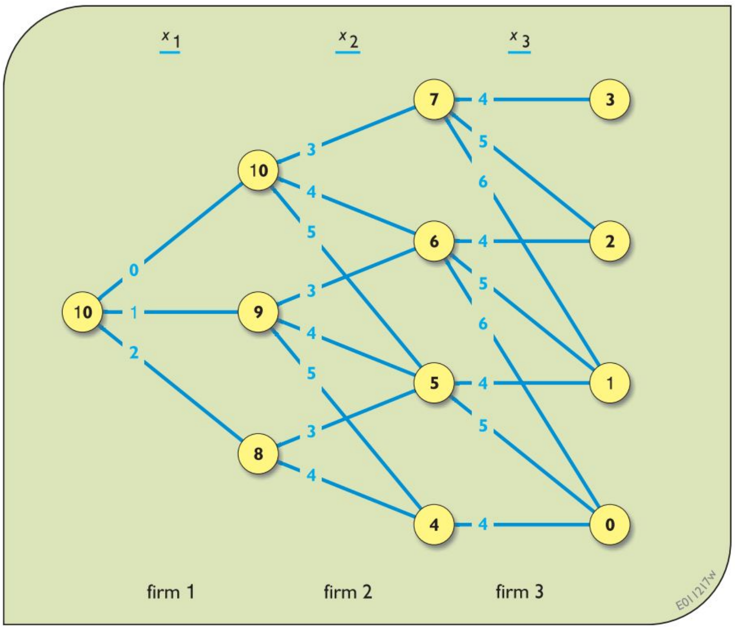


Figure 5.11: A network representing some of the possible integer allocations of water to three water-consuming firms j. The circles or nodes represent the discrete quantities of water available, and the links represent feasible allocation decisions xj [Loucks and Van Beek, 2005].

The links of Fig. 5.11 represent the water allocations. Note that the link allocations, the numbers on the links, cannot exceed the amount of water available, that is, the number in the left node. The number in the right node is the quantity of water remaining after an allocation has been made. The value in the right node, state $S_{j+1}1$, at the beginning of stage $j+1$, is equal to the value in the left node, S_j, less the amount of water, X_j, allocated to firm j as indicated on the link. Hence, beginning with a quantity of water $Q - MIF$ that can be allocated to all users, after allocating X_1 to User 1 what remains is S_2:

$$Q - MIF - X_1 = S_2 \tag{5.1}$$

Allocating X_2 to User 2, leaves S_3, allocating X_3 to User 3, leaves S_4...and so on, till the last user.

$$S_2 - X_2 = S_3 \tag{5.2}$$

Our task is to find the best path through the network, beginning at the left-most node having a state value of Q_0. To do this we need to know the Net Benefits we will get associated with all the links (representing the allocation decisions we could make) at each node (state) for each user (stage).

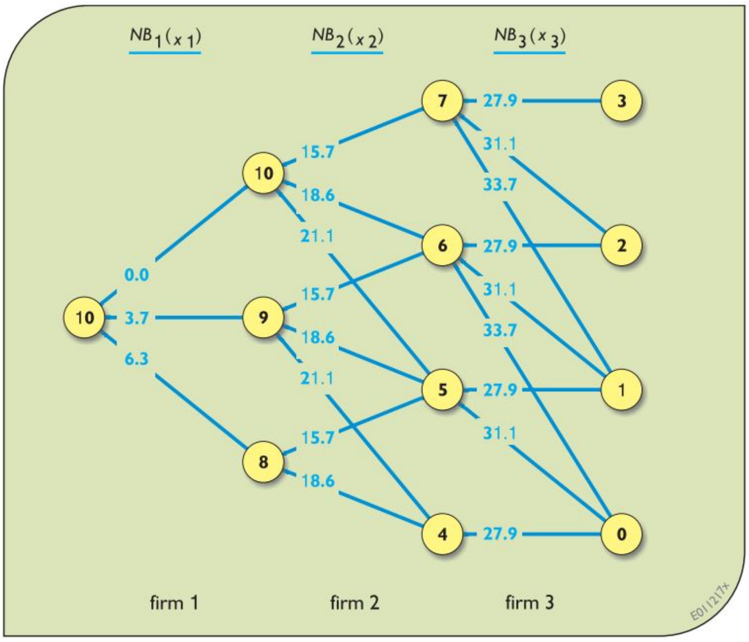


Figure 5.12: Network representing integer value allocations of water to three water-consuming firms. The circles or nodes represent the discrete quantities of water available, and the links represent feasible allocation decisions. The numbers on the links indicate the net benefits obtained from these particular water allocation decisions.

The discrete dynamic programming algorithm or procedure is a systematic way to find the best path through this network. In this type of allocation problem the overall objective is to:

$$Maximize \sum_{j=1}^{n} NB_j(X_j) \tag{5.3}$$

With:
- $NB_j(X_j)$ is the net benefit associated with an allocation of X_j to user j.
- n is the total number of the users

The final general characteristic of the dynamic-programming approach is the development of a recursive optimization procedure, which builds to a solution of the overall n-stage problem by first solving a one-stage problem and sequentially including one stage at a time and solving one-stage problems until the overall optimum has been found. This procedure can be based on a *backward induction* process, where the first stage to be analyzed is the final stage of the problem and problems are solved moving back one stage at a time until all stages are included. Alternatively, the recursive procedure can be based on a *forward induction* process, where the first stage to be solved is the initial stage of the problem and problems are solved moving forward one stage at a time, until all stages are included. The basis of the recursive optimization procedure is the so-called *principle of optimality*: an optimal policy has the property that, whatever the current state and decision, the remaining decisions must constitute an optimal policy with regard to the state resulting from the current decision [Böhme and Frank, 2017].

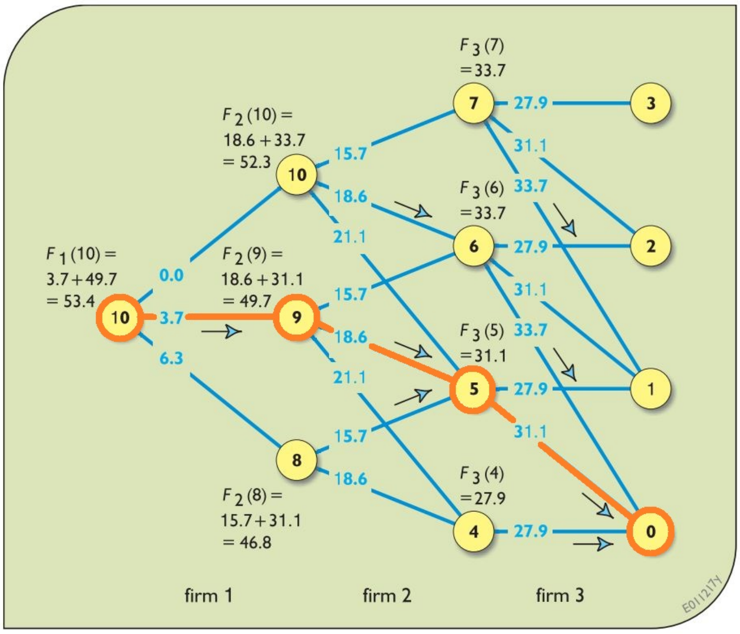


Figure 5.13: Using the backward induction process for finding the maximum remaining net benefits, $F_j(S_j)$, and optimal allocations (denoted by the arrows on the links) for each state in Stage 3, then for each state in Stage 2 and finally for the initial state in Stage 1 to obtain the optimum allocation policy and maximum total net benefits, $F_1(10)$. The minimum flow to remain in the river, MIF, is in addition to the ten units available for allocation and is not shown in this network. The orange highlighted path is the best solution and allocation, giving the greater NB.

5.3.1 Approximation of river system for DPN

In Section 5.1, was explained how it's important the management of water resource in summer period, since there is scarcity of water compared to the demand, for this reason the priority of water allocation is given to irrigation channels, as in effective happens in reality.

For the application of the Dynamic Programming Network, the path of river Adda is approximated to its basic elements for the water allocation. Along river Adda there are many hydropower plants, they withdrawal water for power production and then return the same amount to the river. The plants are responsible for a shifting of the water mass in time and space, but they don't influence the mass balance along the river; for this reason hydropower plants are neglected for the application of Dynamic Programming Network.

Optimization model is applied starting from a certain flow rate delivered by the dam Q_0 $[m^3/s]$ and then distributed between the users. Recalling Section 4.2, there are 4 main tributaries (creek Greghentino, creek Sonna, river Brembo and river Serio) and some groundwater/irrigation discharges contributions in the stretch of river between Rivolta d'Adda and Pizzighettone (in the last part of the stream). As first approximation no tributaries of any type are considered for the model, this means staying on safe side and considering even the possibility of a dry period, during which the water income is very low.

River Adda is reduced only to branches associated with the irrigation channels. The last theme to consider before starting with application of the Dynamic Programming Network is the MIF, it's taken a symbolic value of $20m^3/s$ (almost corresponding to the maximum MIF along the watercourse), this amount of water it's assumed to be subtracted from Q_0 before the application of the model, we are considering the worst case scenario in which tributaries are dry and can't balance MIF. For instance, if for the water allocation it's considered a $Q_0 = 150m^3/s$, actually the water out flowing the dam will be of $Q_0 + MIF = 170m^3/s$. This approximation doesn't influence the downstream water allocation, since this has to be done only with the water exploitable in the river.

Eight control cross-sections with respective hydraulic segments are created along the river path, each one of them is located after a withdrawal and solved using the mass balance equation, as it was modelled in Section 4.3 but with fewer control sections.

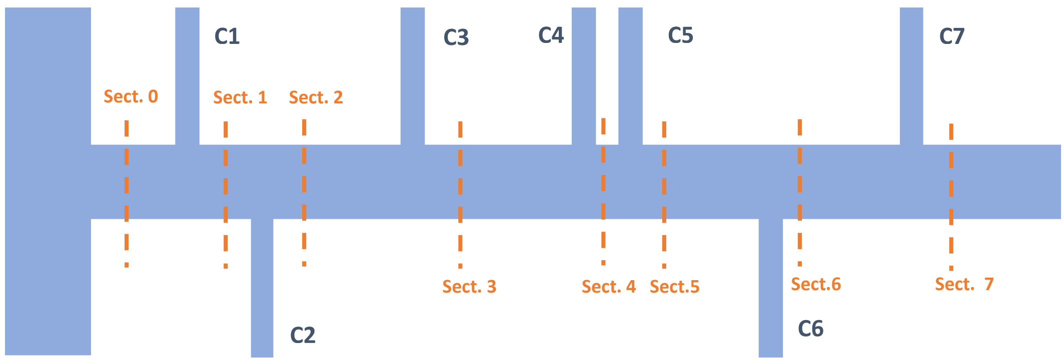


Figure 5.14: Approximation of river Adda for the Dynamic Programming Network

As explained in the previous section, to reduce the complexity of the problem, or rather the size of the network, it's necessary to define a range or fix a number of values for each possible water allocation. Generally in Italy water allocation is done allocating "modulus" of 100 l/s; but since DPN is just an approximation of the best solution, are chosen bigger modulus for the decisions.

The first attempt is done, reducing as much as possible the computational time, with a modulus of $x_j = 10m^3/s$ and 2401 combinations; actually this choice is too approximated since for some channels (i.e. ditch Rivoltana and ditch Vailata) the water concession is less than $10m^3/s$.
The second one with $x_j = 5m^3/s = 5000l/s$, this creates a network of 233280 feasible and not-feasible combination for water allocation (some of them aren't possible because the total amount of water "theoretically" allocated is bigger than Q_0).

To further refine the allocation possibilities it's chosen $x_j = 1m^3/s$, this creates more than 20 millions combinations with a computational time over a day, too much for an optimization model.

It's tried a trade-off for DPN of $x_j = 2m^3/s$, a quite good compromise between modulus' size and computational time. There are in total more than 5105987 combinations, considering both possible and not, with a computational time about 7 hours, still to high for an optimization model in which will be tested many policies.

The final choice is to use a modulus of $x_j = 5m^3/s$, considering that this is just an approximation and simulation is necessary to refine the final result.

5.3.2 Net Benefit Curves for irrigation

The best solution in Dynamic Programming, and in others Optimization Methods, is the one that maximizes the total Net Benefit (NB). It's very difficult to build net benefit curves for irrigation channels, they come from a complex agro-economic analysis that involves not only the flow rate provided, but also many other specific factors like precipitations, temperatures, climate in general, crop types, irrigation techniques...

The estimation of crops irrigation requirement depends on the hydrological budget, in which are considered water consumption by the plants for evapotranspiration, infiltration, superficial runoff, percolation losses, capillary rising flow and soil moisture.

Irrigation demand I is the water volume necessary to balance the deficit between potential evapotranspiration and rainfall, following:

$$\Delta U = I - ET_p + (P_e - RO) - DP + CR \tag{5.4}$$

with:

- ET: effective evapotranspiration
- P_e: effective precipitation
- RO: superficial runoff
- DP: deep percolation
- CR: capillary rise
- ΔU: soil moisture variation concerned by crops' roots, in the interval of time considered

The effective evapotranspiration is computed starting from the potential evapotranspiration and adjusting it in function of colture's growth [Gandolfi, 2003].

$$ET = K_S K_C ET_0 \tag{5.5}$$

- ET_0: reference evapotranspiration, only depending on climatic areas
- K_C: characteristic crop factor
- K_S: stress coefficient, depending on management and environmental factors that reduce evapotranspiration rate below the potential one

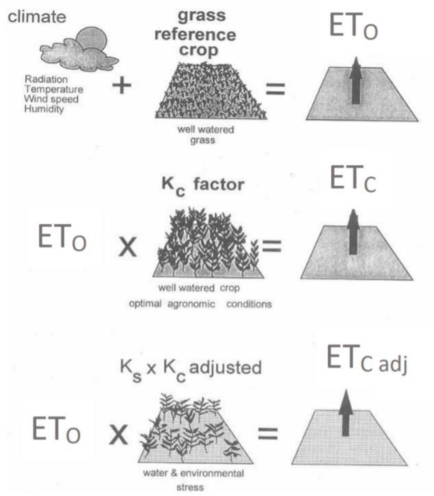


Figure 5.15: Computation of evapotranspiration following "FAO -Irrigazione e Drenaggio n.56"

The irrigation demand varies in time and space, principally basing on the weather and the crops type. Considering only the part concerning agriculture, namely neglecting elements like navigation needs, thermoelectric plants, municipal and industrial water demands; the irrigation requirement can be taken as the main element for the construction of NB curves.

Concession contracts managed by Consorzio dell'Adda are stipulated ahead between irrigation users and Italian government, basing on agricultural needs for the various irrigation districts. Starting from concession flow rates, with the help of Professor. Eng. Claudio Gandolfi have been outlined a draft curve for irrigation net benefit, since an unique curve model doesn't exist.

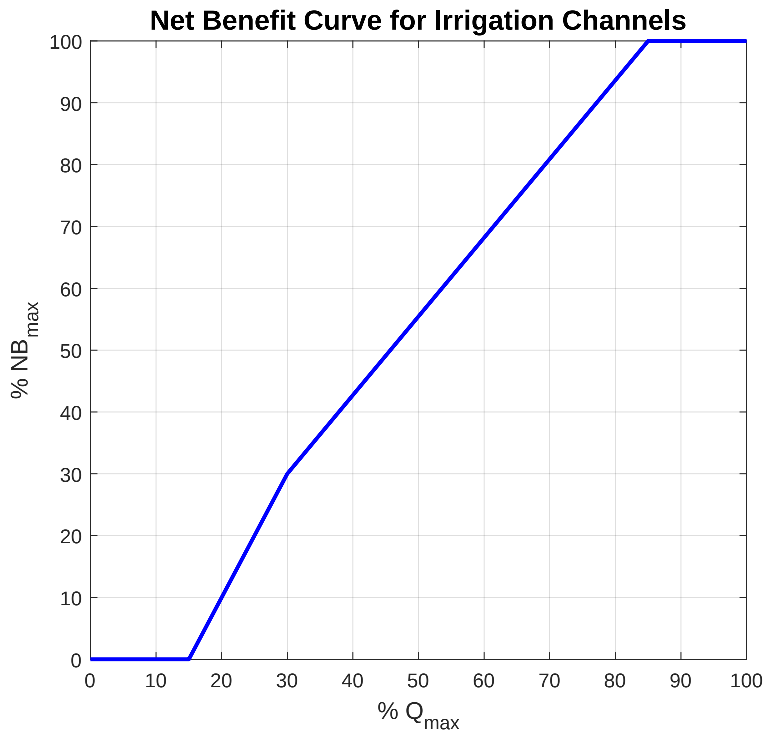


Figure 5.16: Draft of net benefit curve for irrigation channels (in cooperation with Eng. Gandolfi)

Where:
- Q_{max} is the concession flow rate for a given irrigation channel
- NB_{max} is the maximum net benefit possible for a given irrigation channel

The shape choice depends on the fact that until there is enough water, crops grow flourishing even without the optimal amount of water from irrigation (it can be compensated by precipitations, soil moisture, aquifer contributions...), this threshold can be placed at 85% of water concession flow rate. Then the relationship can be considered quite linear. As water scarcity increases the damage will increase exponentially, reducing earnings related to the agricultural production, below 15% of maximum irrigation flow rate it's almost impossible to grow something.

In reality, the benefit deriving from agriculture is extremely variable and influenced by precipitations over all the region. The competition and trade-offs between irrigation users are difficult to be monetized since each user is subjected to the contracts with Consorzio dell'Adda. For this reason is taken the same NB curve for each user and no weights are applied; even though there are some "main channels", the management authority is responsible for ensuring the fair distribution of water between the users.

5.3.3 Development and results

The use of Dynamic Programming Network is functional for the purpose since it simplifies the problem and well approximate Adda river system. There is no need for a continuous and convex objective function and there is the possibility of easily change NB curves, applying weights or other coefficients.

It is done water allocation with $Q_0 = 100m^3/s$, plus $MIF = 20m^3/s$ it's considered a total amount of water delivered by the dam of $120m^3/s$, a case of poor outflow during summer period so when it's fundamental a correct water management. Since it's used a "modulus" $x_j = 5m^3/s$, the canals concessions are approximated by excess till a multiple of 5, in the computation there are 82737 possible configurations.

Note: The objective function to be maximized is Total Net Benefit, so the sum of NB deriving from each irrigation user of the system. The curve created is associated to percentages of NB and not to a certain amount of money; clearly maximum NB of channel Muzza will be greater than maximum NB of one other secondary channel like ditch Vailata, monetary talking, but for now this is irrelevant for the optimization, since concession contracts behind don't involve any priority in summer period.

The computation gives some possible "optimal" combinations with the same Total NB = 558% , note that the Maximum Total NB in this case is 700%, so the Total NB for configuration 1 is 79% Max Total NB; below the example of one configuration:

Table 5.1: DPN configuration 1: modulus $x_j = 5m^3/s$ and Total NB = 79% Max Total NB

	C1	C2	C3	C4	C5	C6	C7
$X_j[m^3/s]$	10	30	10	15	10	0	25
NB_j	100%	100%	100%	87%	100%	0%	71%

As shown in Tab. 5.1, the total net benefit is maximized, but isn't given any importance to channel Muzza (the greatest one) that remains completely dry.

For the second DPN configuration it's guaranteed a minimum flow rate in channel Muzza of $50m^3/s$, necessary all over the year for navigation and for thermoelectric plants. Total NB = 447%, that is less than the previous configuration, so it's no more the "optimal" solution for what concern the total earning from the channels.

Table 5.2: DPN configuration 2: modulus $x_j = 5m^3/s$ and Total NB = 64% Max Total NB

	C1	C2	C3	C4	C5	C6	C7
$X_j[m^3/s]$	10	0	10	20	10	50	0
NB_j	100%	0%	100%	100%	100%	47%	0%

Configuration 3 is done ensuring a minimum flow for principal channels: C. Martesana ($10m^3/s$), C. Retorto ($10m^3/s$), C.Muzza ($30m^3/s$) and C. Vacchelli ($10m^3/s$); the amount of water minimum for channel Muzza has been decreased since $50m^3/s$ is half of the water delivered by the dam, too much considering the presence of other three quite important canals. Total NB = 475%.

Table 5.3: DPN configuration 3: modulus $x_j = 5m^3/s$ and Total NB = 68% Max Total NB

	C1	C2	C3	C4	C5	C6	C7
$X_j[m^3/s]$	10	10	10	15	10	30	15
NB_j	100%	27%	100%	87%	100%	22%	39%

Many other configurations are tried, it is noticed that whenever there isn't an imposition for a minimum amount of water in irrigation channels, the tendency of DPN (assuming NB curves described in Subsection 5.3.2) is to set to zero some channels with the aim of favoring other ones maximizing their net benefit.

Following the policy of respecting the contracts without favoring any channel, are created new configurations based on the *principle of proportionality*, trying to keep the same %NB for all the irrigation channels. The first is done with a "modulus" = $5m^3/s$, Total NB = 336%. The second one with $x_j = 2m^3/s$ less than the previous so as to be more precise and spread more the water, Total NB = 314%, paradoxically uniforming NB_j the Total NB decreases because no channel receives concession flow rate.

Table 5.4: DPN configuration 4: modulus $x_j = 5m^3/s$ and Total NB = 48% Max Total NB

	C1	C2	C3	C4	C5	C6	C7
$X_j[m^3/s]$	5	10	5	10	5	50	15
NB_j	55.5%	27%	55.5%	55.5%	55.5%	47%	40%

Table 5.5: DPN configuration 4': modulus $x_j = 2m^3/s$ and Total NB = 45% Max Total NB

	C1	C2	C3	C4	C5	C6	C7
$X_j[m^3/s]$	4	16	4	8	4	46	18
NB_j	43%	50%	43%	43%	43%	43%	49%

Table 5.6: DPN configuration 4": modulus $x_j = 1m^3/s$ and Total NB = 45% Max Total NB

	C1	C2	C3	C4	C5	C6	C7
$X_j[m^3/s]$	4	15	4	9	4	47	17
NB_j	43%	46%	43%	49%	43%	44%	46%

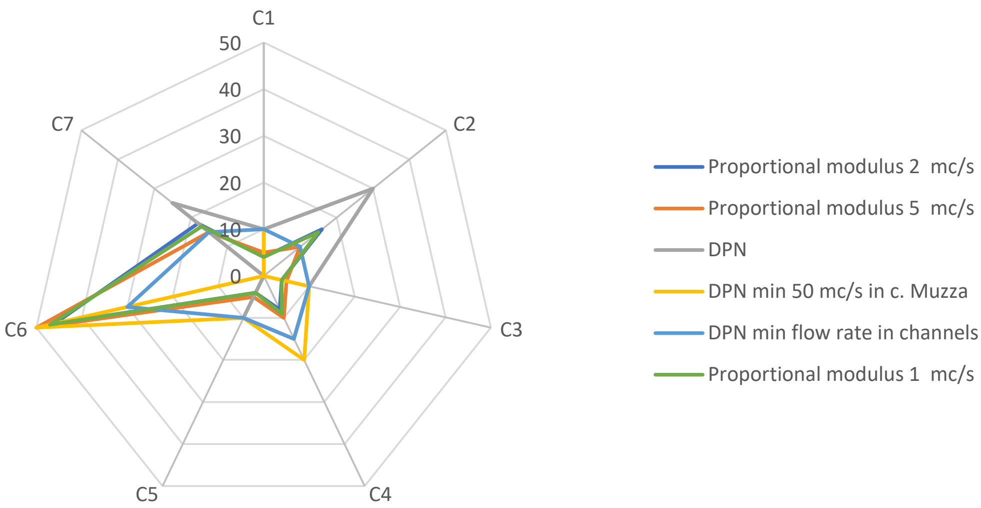


Figure 5.17: DPN configurations for water allocation in irrigation channels

Different policies can be adopted, any of them favoring different aspects such as proportionality, the maximum total NB, the priority for the greater channels, the common good ecc... There is not an unique solution for water allocation: the one that at a first sight can be considered the "best one" such as configuration 1, in reality has many limits of practical or political nature; other configurations like configuration 4 that apparently have a very low Total NB, can be considered more interesting solutions for the proper functioning of the irrigation districts. We always need to take a good hard look at the results obtained, that must be refined with simulation methods before being applied in a real allocation policy.

From an agronomic perspective, talking about water allocation in a single day is quite useless since crops don't grow from one day to another, and at the same time don't die if for one day don't receive the necessary amount of water. Let's say: *"Plants on the balcony don't die, if I don't give them water for one day."*
Agriculture is based on seasonal planning and on climate, what is important is to have, or at least to hope to have, a certain amount of water available for crops.

In a wider point of view water management is no longer to be considered daily and based on flow rate delivered by the dam, but seasonal and focused on volumes accumulated during this period. The daily optimization for water allocation isn't enough in a system where there is an upstream regulation dam and many irrigation districts downstream; a complete optimization has to start before and to be planned for a longer period, involving regulation strategies for the reservoir.

For these reasons in the next Chapter will be discussed about a new Optimization Model for the whole system, founded on future projections of cumulative volumes from inflows.

CHAPTER 6

OPTIMIZATION MODEL: PROJECTIONS FOR FUTURE INCOMING VOLUMES

"The science of real-time hydrologic forecasting has reached the point where significant advances can be made to provide improved information for water managers. A water resources forecast may range from the estimation of the stage or discharge of a river for the next one or two days to the prediction weeks or months into the future of quantities such as volume, maximum flow, minimum flow, and time until an event occurs. As the duration of the forecast period increases, the level of uncertainty in the forecast also increases. Information about uncertainty is an important part of a water resources forecast. Thus, water resources forecasts are probabilistic statements about the future. They are particularly useful in decision making when uncertainty is considered explicitly" [John and Schaake, 1991].

The case of study, that is Lake Como and river Adda basin, is very complex to be modeled and do future forecasts about its hydrological parameters is even more. Uncertainties are many in this water catchment, starting from the internal agreements between upstream hydropower plants in Valtellina and Lake Como management authority, as well as terms among irrigation users and other local figures downstream Olginate's dam, groundwater behavior still has to be fully comprehended and also the climate is particular over the whole region and difficult to be modeled. Unforeseeable and violent flood event frequently occur, especially when there are lots of precipitation in the Alps; like in the other side very long dry period can characterize the area, putting in strain all the water supply system. Optimization models work on what is best for a particular situation in a particular moment characterized by a series of known parameters and equations. It's hard to deal with uncertainties while programming this type of models, especially when uncertainties involve many different aspects of the river system.

The idea isn't to pretend to design a new optimization model for Lake Como regulation and optimal water management, as introduced in Chapter 1 many studies have be done in the past and also new projects like Adda2020 are in development. Taking inspiration from a method of forecasting from the website *www.laghi.net* by Hortus Srl, was born the idea of proceeding in another way with the problem of optimal regulation. The objective is to create a tool useful for the management authority, Consorzio dell'Adda, and easy to be used, for planning the regulation of Lake Como in a quite long period, in particular in summer one.

Again, as was explained in Section 1.1, generally during the year about 100 regulation maneuvers are made at the dam site. Numerical solutions that aim to maximize/minimize a multi-objective function, they typically correct the deliver outflow frequently to be as close as possible to the maximum/minimum. Moreover, optimization model using weather forecasts can vary their solution any minute, according to the changes in weather patterns. A regulation policy that aims to keep for many days consequently the same opening for the sluice gates of the dam is more feasible and applicable to the real case of Lake Como. Dam's opening changes should be done when needed, and not for unnecessary and futile adjustments of delivered outflow.

6.1 FORECASTS OF FUTURE INFLOWS

The planning of a new regulation policy for the Lake is based on forecasts of future inflows volumes starting from historical data series. There are many methods applicable, each of them with some uncertainties related. In this elaborate are applied and compared three different approaches: empirical, probabilistic "a priori" and probabilistic "a posteriori".

Given a certain day during the year, the regulation policy will be based on cumulative volumes of inflows till that day and on projections of future inflows starting from that date.

Firstly are observed data series of inflows in $[m^3/s]$ till that day , let's say for instance 09/04/2019 (day 100 of the year) that is almost one month before the beginning of summer season. Thanks to Fig. 6.1, flood events and period of drought can be distinguished and in general the trend of the year.

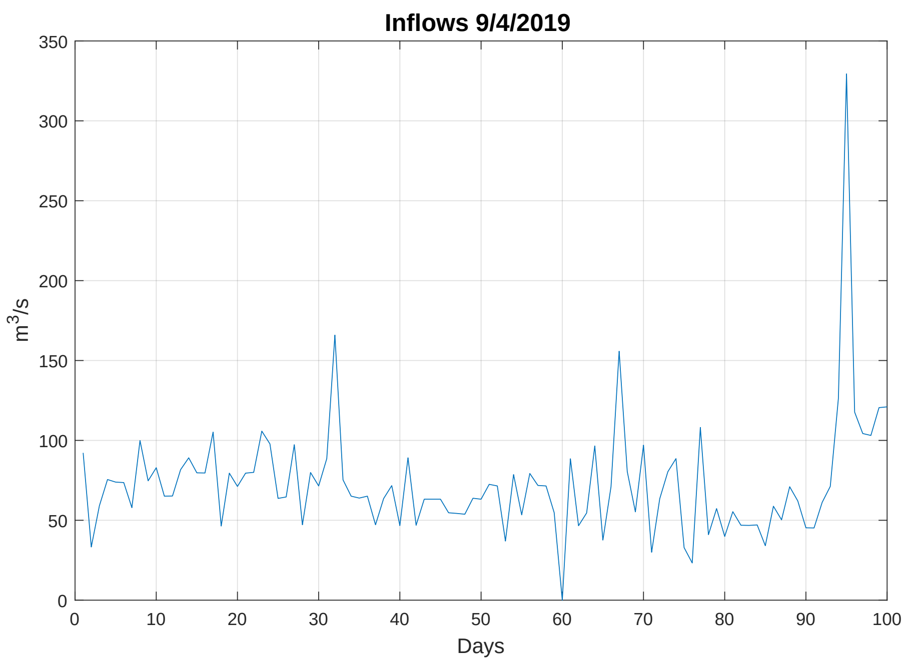


Figure 6.1: Inflows 09/04/2019 $[m^3/s]$

Are computed in-going cumulative volumes, they are fundamental for the comparison with historical data series, indeed it's more useful think in summed up volumes than in daily flow rates, remembering that what we need to know is the total amount of water that during the year will be exploitable for agriculture.

$$W_i(t) = \int_0^t Q_i(\tau) d\tau \quad [m^3] \tag{6.1}$$

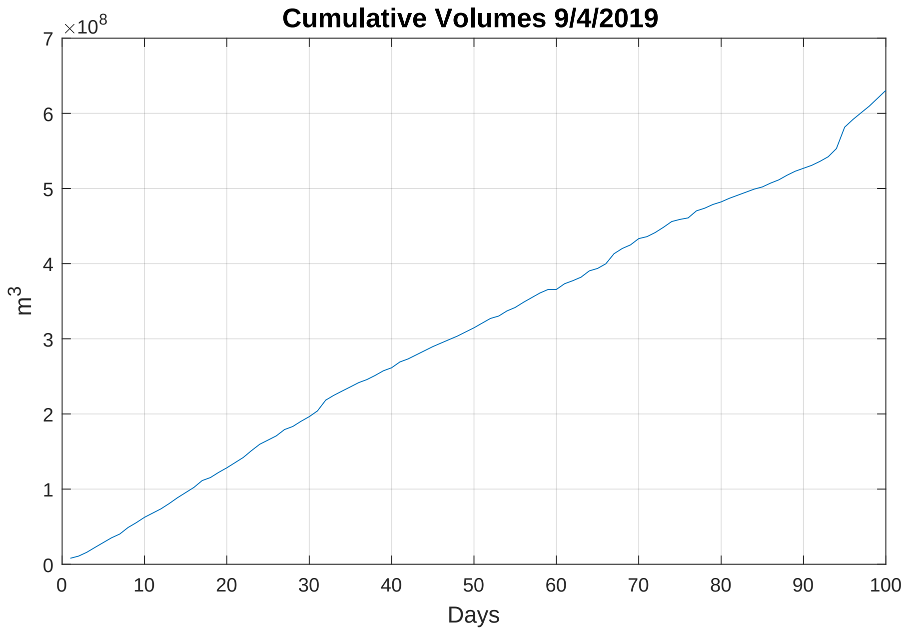


Figure 6.2: Cumulative volumes of inflows 09/04/2019 [m^3/s]

Historical data series last 74 year starting from 1946, recalling that Olginate's dam was finished in 1944). At the end of the year cumulative volumes of inflows can differ even for four-five orders of magnitude, this means that we can't pretend to crop and have the same production from agriculture in every year.

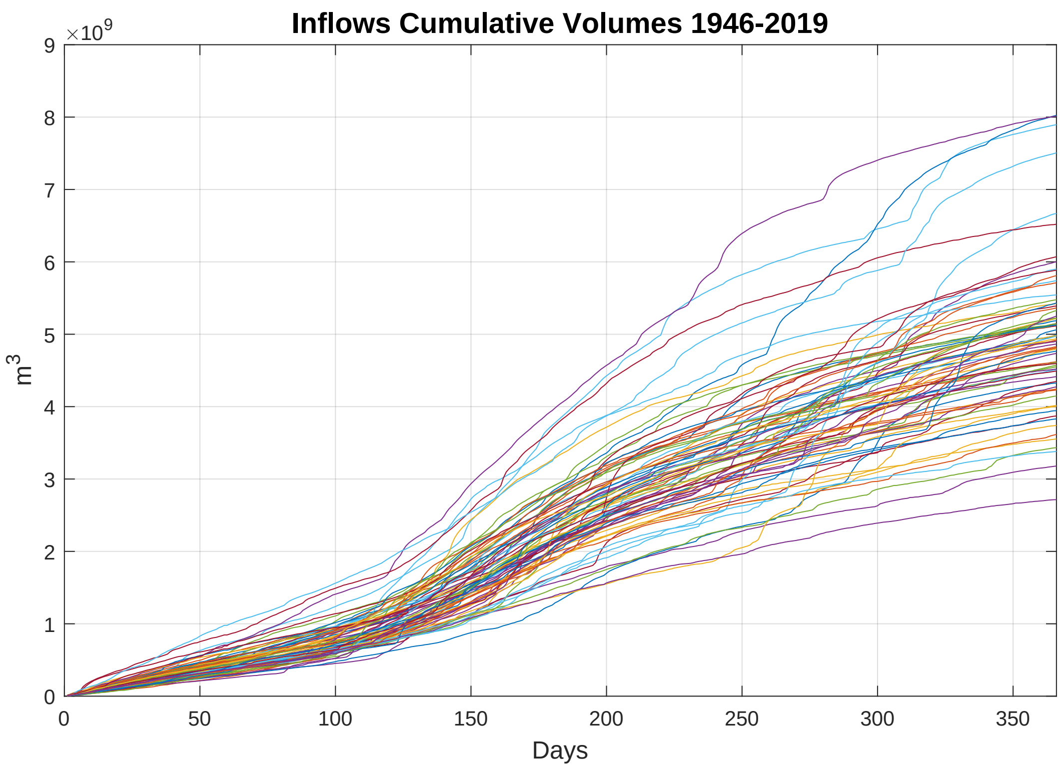


Figure 6.3: Cumulative volumes of inflows from historical series 1946-2019

6.1.1 EMPIRICAL MODEL: LEAST SQUARES METHOD

The first approach it's based on knowledge of inflows data series. The main idea is that given a certain day and done the cumulative volumes of inflows till that day, the cumulative volumes of inflows for the rest of the year will have a similar behavior to the ones from the historical series which have more similar trend in the part of the year already observed, for what concerns the cumulative volumes of inflows.

This concept it's justified by the fact that in nature it's possible to identify more dry and rainy years; in general a drier year will follow a behavior like the others dry years, even though there can be some exceptions. Therefore, in order to forecast the plausible total amount of water from inflows, it's useful to do a first screening identifying which type of year is current. In Fig.6.4 rainy years and dry ones are distinguished, thanks to the sum of *mm* of precipitation along the year in Lombardia Region.

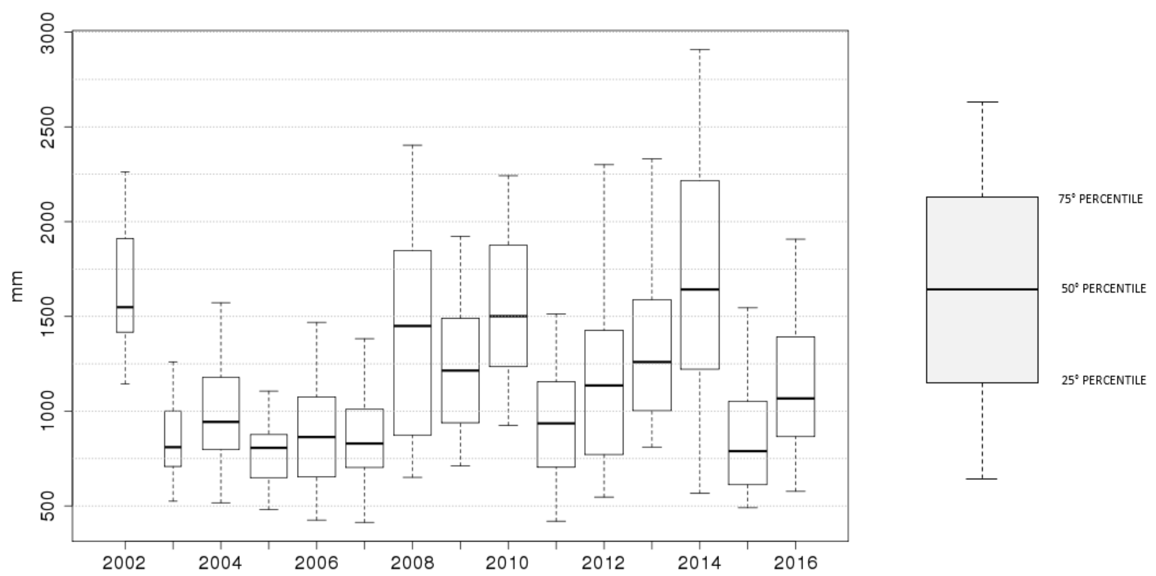


Figure 6.4: Box plot of the sum of precipitation registered in the year from each station of the monitoring network of ARPA Lombardia (www.arpalombardia.it).

One other important aspect to keep in mind is that river Adda water catchment it's located in the north of Lombardia, with a great area in the Alps. Here the snowy precipitation in winter it's huge and plays a very important role for the amount of water involved, that later in spring period will reach Lake Como as inflow.

It's very complex to assess properly the amount of water related to the snow pack and the snow cover over the region, which can vary in depth and density even within the same valley. It's also difficult to forecast when the snow pack will be converted into runoff, since the snow melting period can vary from the end of March till mid-June. A fraction of the snowmelt, that would naturally flow to the lake, is captured by several artificial hydropower reservoirs located in the upstream part of the catchment; the release of this quantity of water it's ruled by the energy companies involved, so it's even more complicated to predict when the water from the snowy precipitation will reach Lake Como.

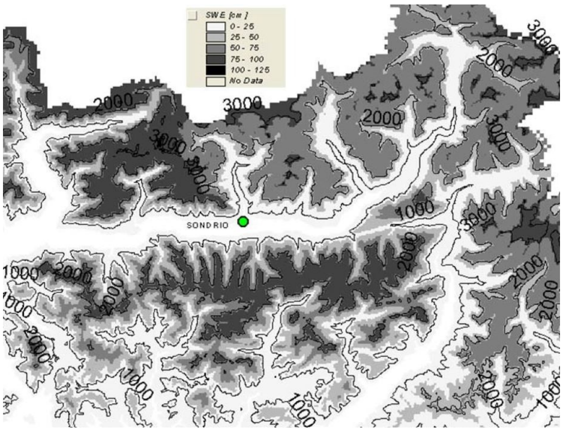


Figure 6.5: Distributed SWE in Valtellina Valley area in 1985

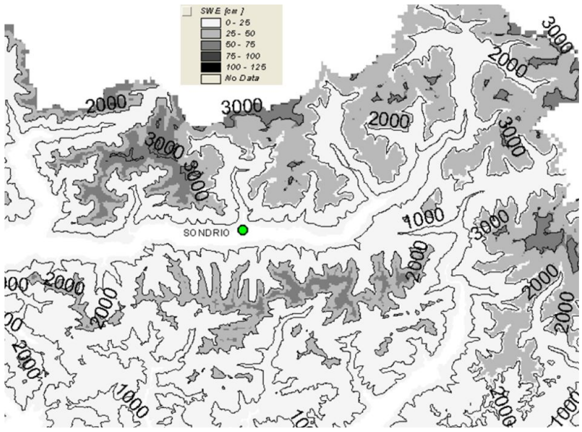


Figure 6.6: Distributed SWE in Valtellina Valley area in 2001

The snow water equivalent (SWE) describes the equivalent amount of liquid water stored in the snow pack, it's associated to the snow density, the water density and the snow depth [Bocchiola, 2018].

For all these reasons the search of the most similar years from the historical data it's done through the comparison of cumulative volumes from the beginning of the year and not just for "some days before" the starting date of the forecasts. In this way for the choice of similar years it's considered not only if the year is dry or not, but also if the snow melting has already started and involving which amount of water.

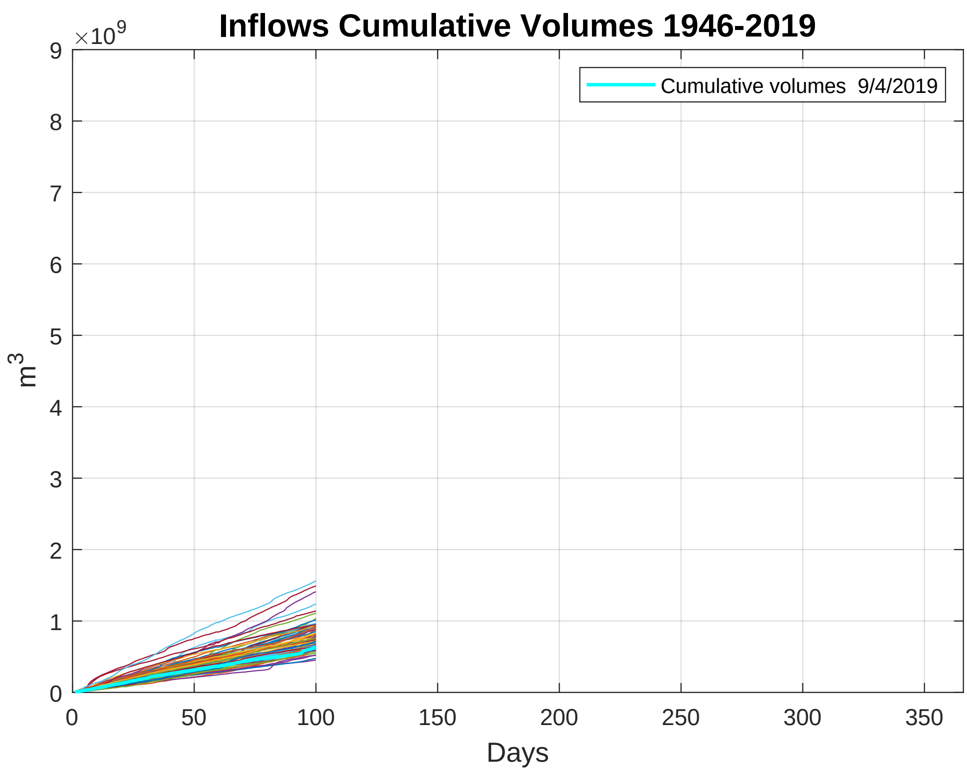


Figure 6.7: Cumulative volumes of inflows till 09/04/2019, starting day of forecasts

Least Squares Method is employed for the comparison between reference cumulative volumes of inflows and data series. The method of least squares is a standard approach in regression analysis to approximate the solution of over determined systems by minimizing the sum of the squares of the residuals made in the results of every single equation. The most important application is in data fitting. The best fit in the least-squares sense minimizes the sum of squared residuals S_n [Miller, 2004].

$$S_n = \sum_{i=1}^{n} r_i^2 = \sum_{i=1}^{n} (y_n - (ax_n + b))^2 \tag{6.2}$$

Generally y_n represents the observed data and $ax_n + b$ the fitting curve. In our case the cumulative volumes of the reference year are considered as the observed data and the cumulative volumes from historical data as the "fitting" one. It really isn't happening any data fitting, but it's just a way to measure which year is more similar to the one of reference.

$$S_n = \sum_{i=1}^{n} r_i^2 = \sum_{i=1}^{n} (V_{ref_i} - V_{n-year_i})^2 \tag{6.3}$$

Where:

- S_n is the sum of the squared offsets for the n_{th} year in the historical series with respect to the year of reference. - V_{ref_i} is the cumulative volume of inflows in day i in the reference year. - V_{n-year_i} is the cumulative volume of inflows in day i in the n_{th} year in the historical series.

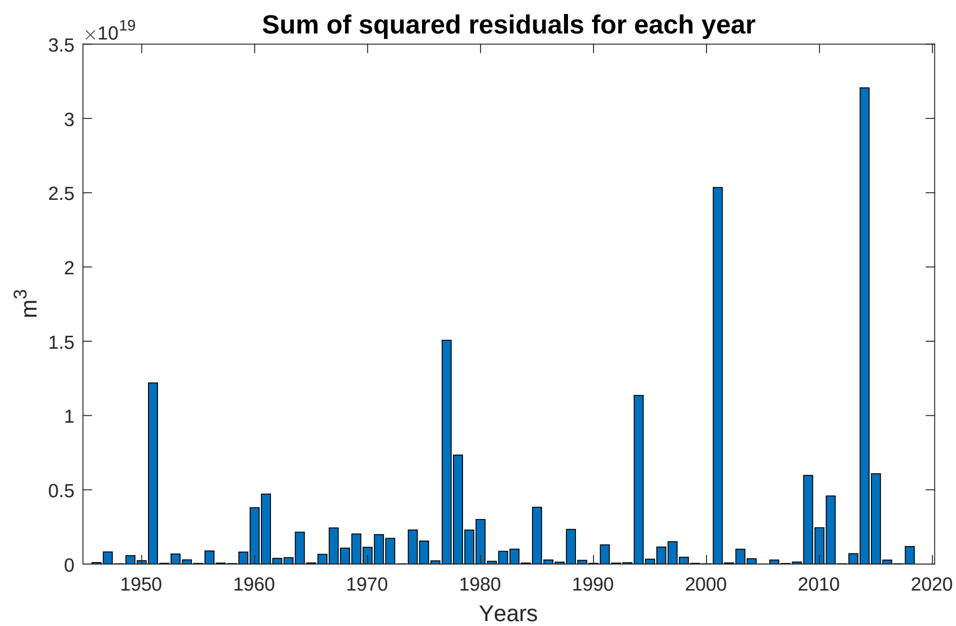


Figure 6.8: Sum of the squared residuals for each year [m^3]

It's chosen a numerosity of 15 for the sample of similar years, the ones with the minimum S_n will be used for the future forecast of inflows. From 1946 till 2019 there are 74 years, so 15 years are almost the 20% of the total population.

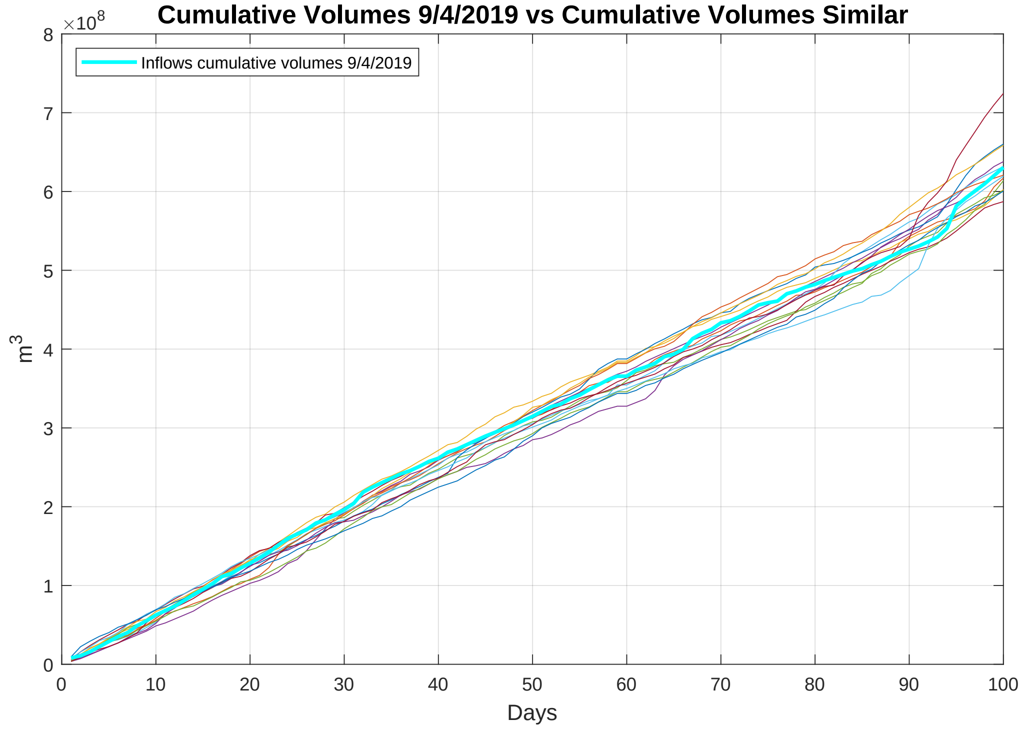


Figure 6.9: Cumulative volumes of inflows until 9/4/2019 (day number 100 in the year) vs cumulative volumes of inflows of the 15 similar years until day 9/4

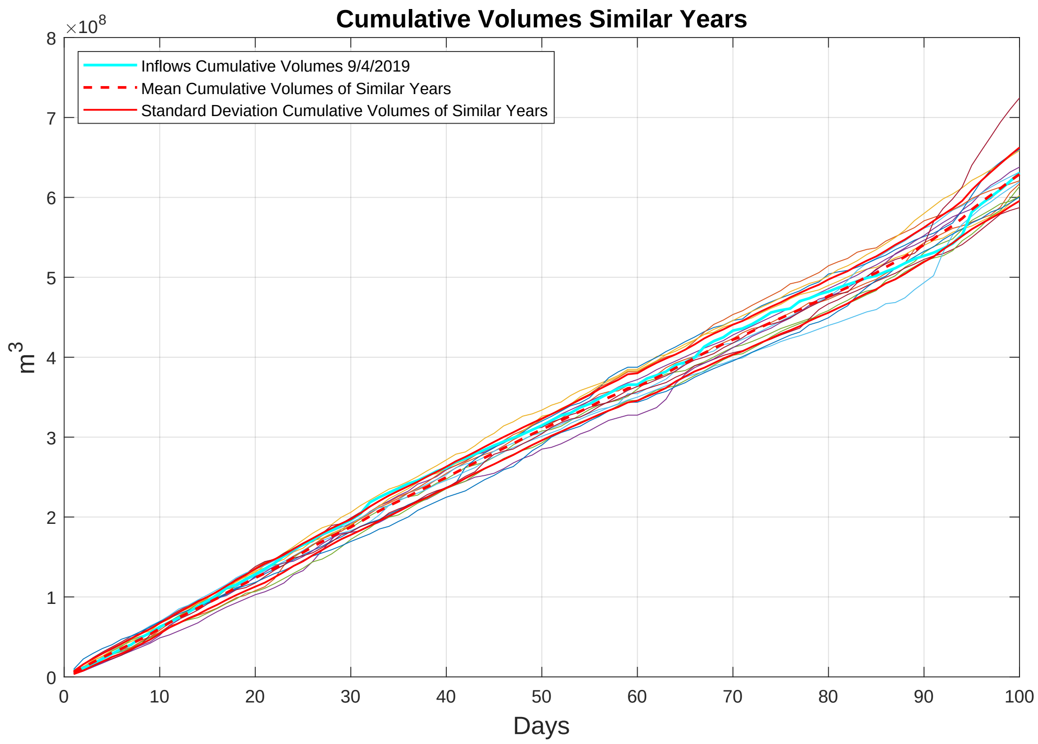


Figure 6.10: Cumulative volumes of inflows of similar years, their mean and standard deviation

Once individuated the sample of similar years, the cumulative volumes of inflows till the end of the year and their mean and standard deviation are plotted.

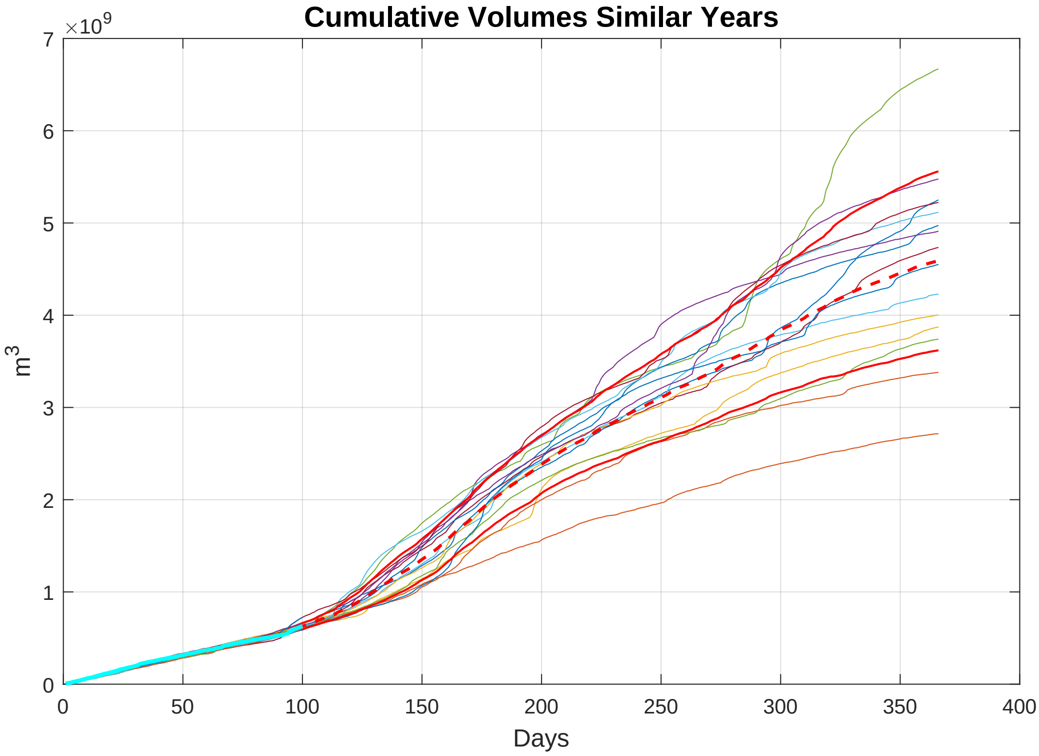


Figure 6.11: Cumulative volumes of inflows of similar years with mean and standard deviation till the end of the year

The standard deviation makes a "red band" inside which it's very probable that the future cumulative volumes will stay, for short term inflows the probability increases and the forecast is more reliable.

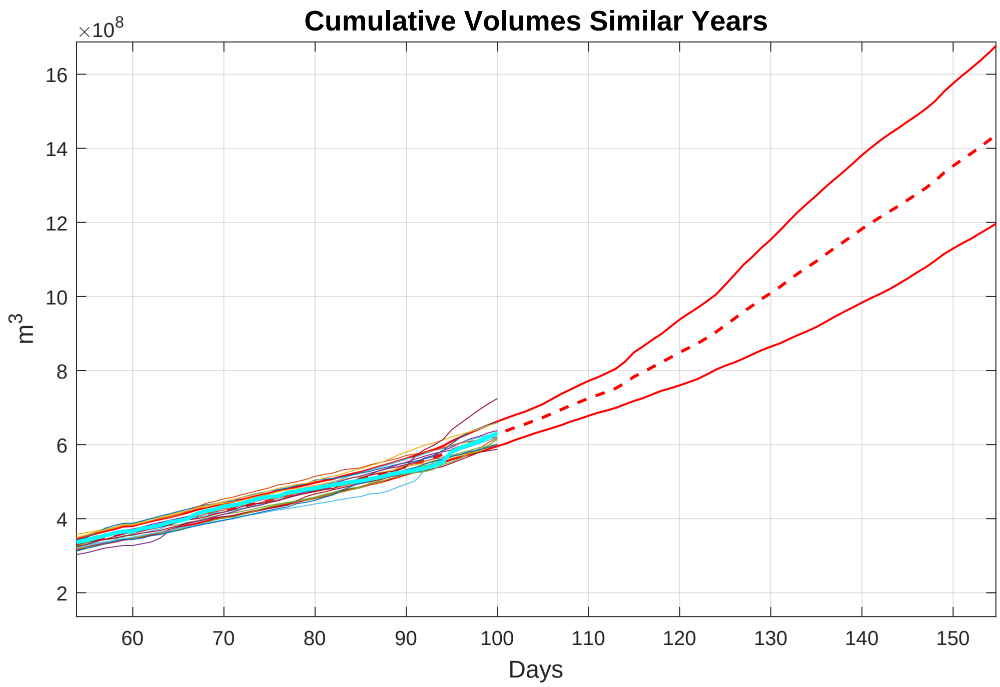


Figure 6.12: Cumulative volumes of inflows of similar years "red band", zoom in short term

CHECK OF THE EMPIRICAL MODEL

It is done a check for the method applying it for several random days from the historical series. Its validity is tested by plotting the effective cumulative volumes of inflows for the whole year and by comparing them with the "red band" of forecasts.

Below are shown some examples for the application of the method: the method works quite well since the cumulative volumes have the tendency to stay inside the "red band" given by the standard deviations, especially for the short term.

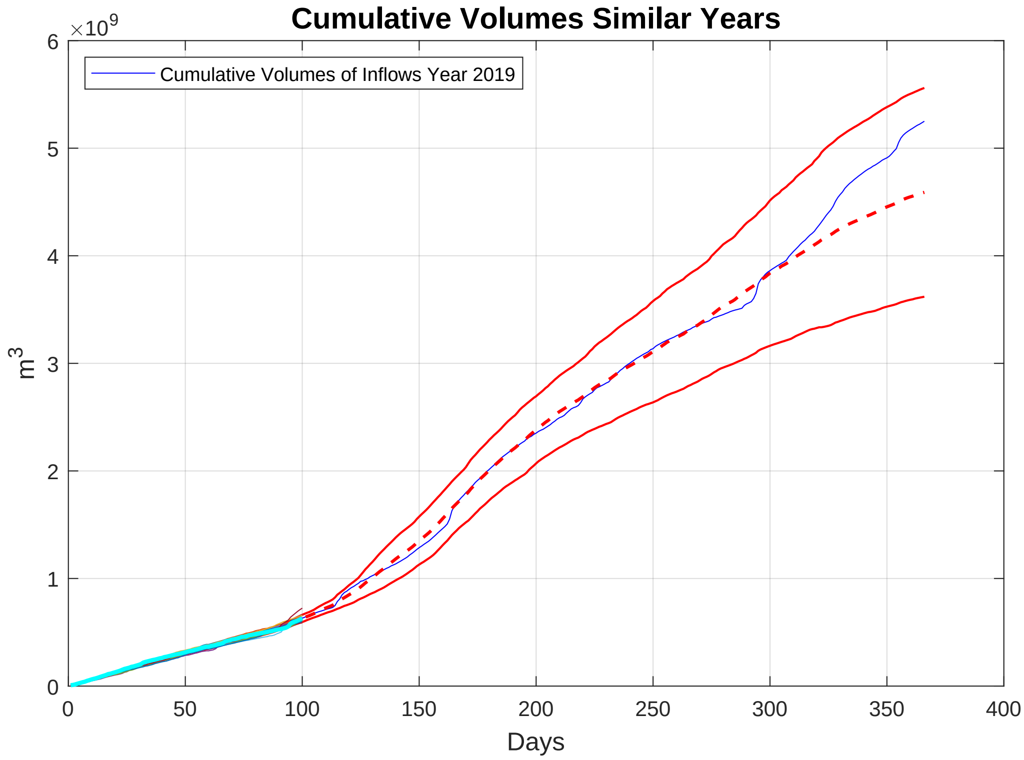


Figure 6.13: "Red band" given by the cumulative volumes of inflows of similar years and year 2019 cumulative volumes of inflows

Note: each time it's inserted a new start date for the forecasts, the program computes the sample of the similar years with the least squares method applied on the cumulative volumes till that day. Therefore the "red band" updates anytime changes the starting date, this makes the projection even more reliable for the short term since it's constructed step by step incorporating all the information till that moment.

Forecasts made on the base of short time series of cumulative volumes, for instance with a starting date in the beginning of February, are not really precise since the data used for finding the most similar years are not enough to make a thorough analysis.

The method it's based on the historical data and finds the "red band" basing on the cumulative volumes given by them, if the year of study it's an non-ordinary year, with huge extreme events, probably the cumulative volumes curve will go outside the prevision. This is important to keep in mind especially for the autumn period when the flood events can be very dangerous and unpredictable (as in Example 3).

Example 1: Starting date of the forecasts 09/08/1966

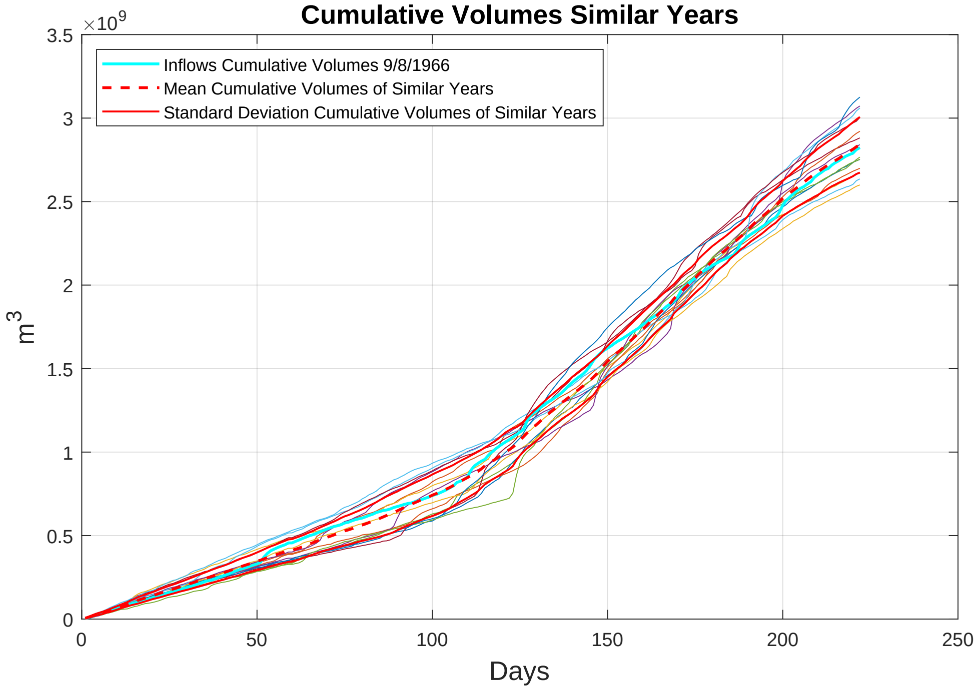


Figure 6.14: Example 1: Cumulative volumes of inflows of similar years with their mean and standard deviation

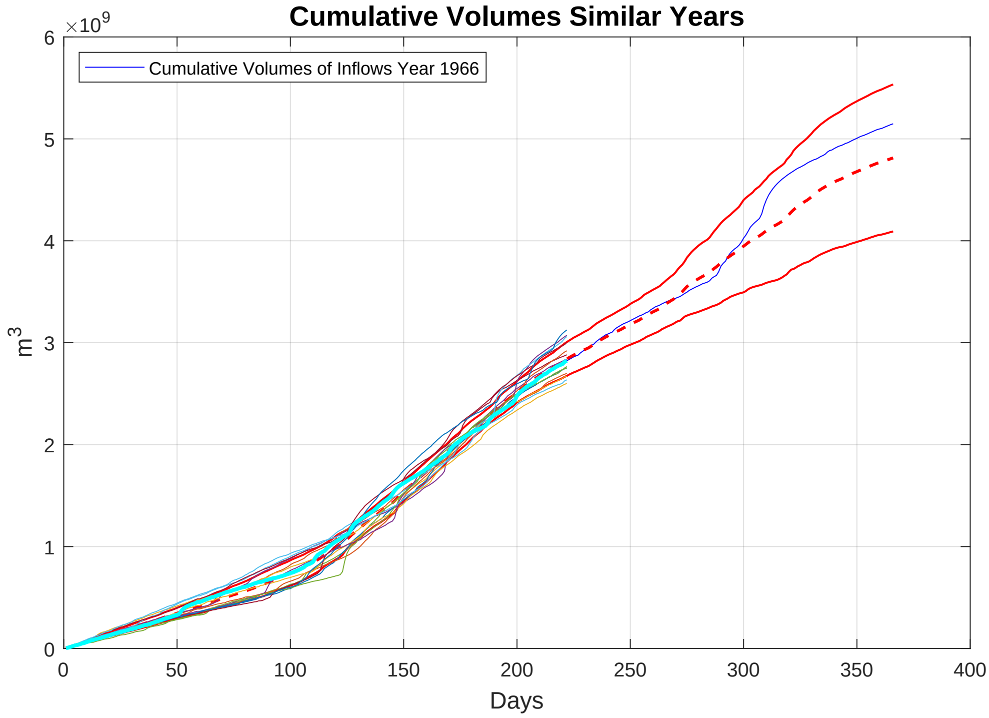


Figure 6.15: Example 1: "Red band" given by the cumulative volumes of inflows of similar years and year 1966 cumulative volumes of inflows

Example 2: Starting date of the forecasts 25/06/1971

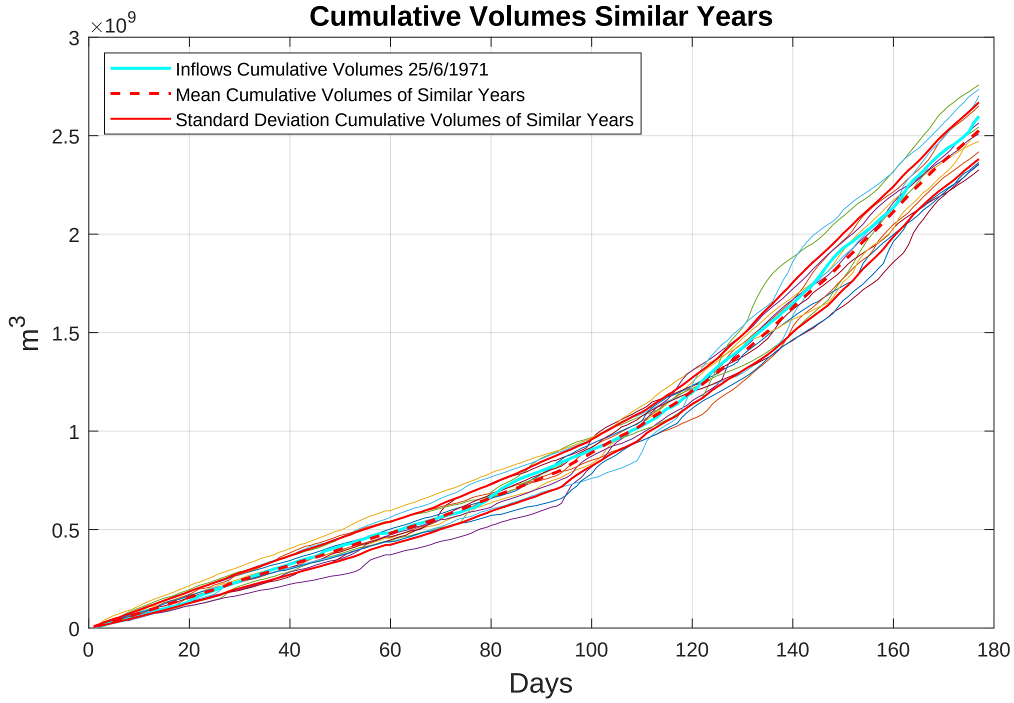


Figure 6.16: Example 2: Cumulative volumes of inflows of similar years with their mean and standard deviation

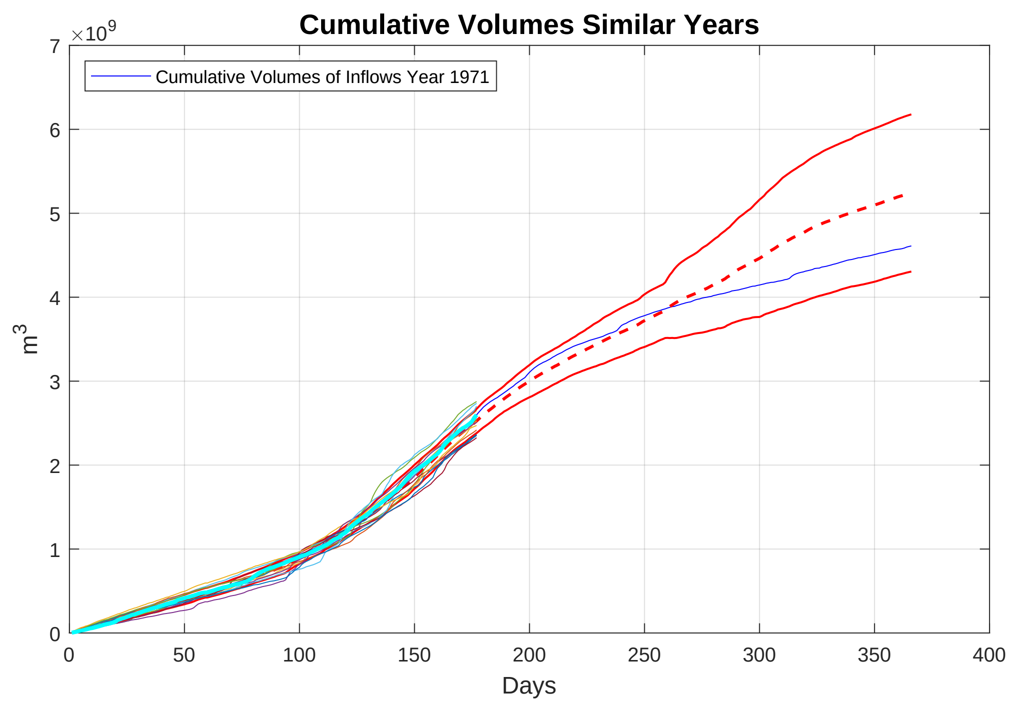


Figure 6.17: Example 2: "Red band" given by the cumulative volumes of inflows of similar years and year 1971 cumulative volumes of inflows

Example 3: Starting date of the forecasts 24/05/2002

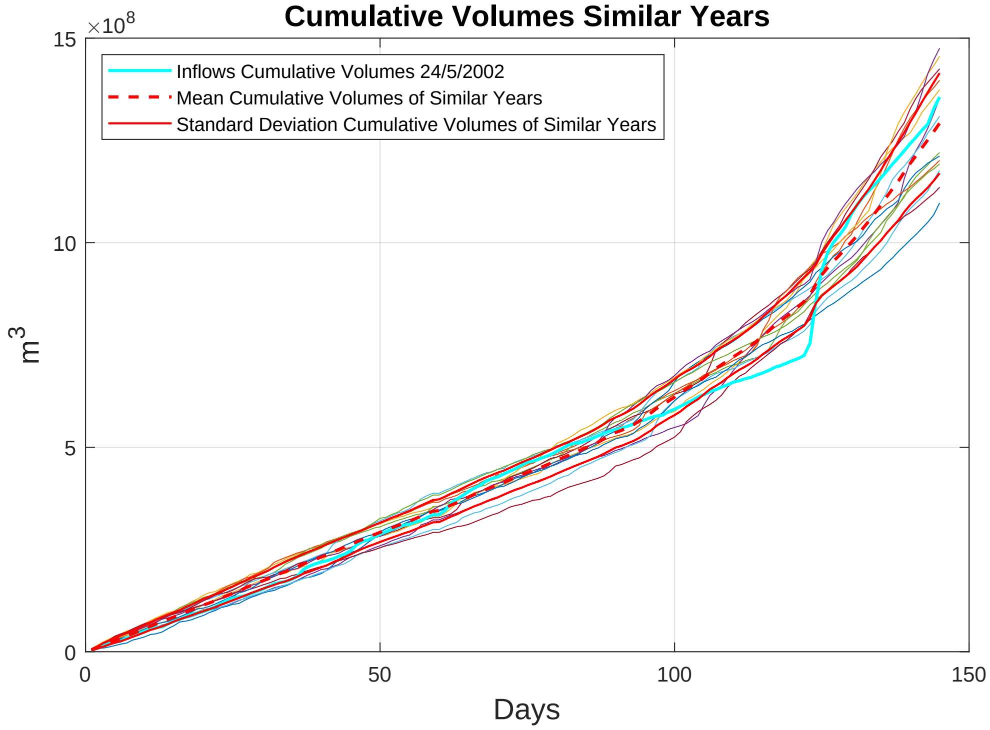


Figure 6.18: Example 3: Cumulative volumes of inflows of similar years with their mean and standard deviation

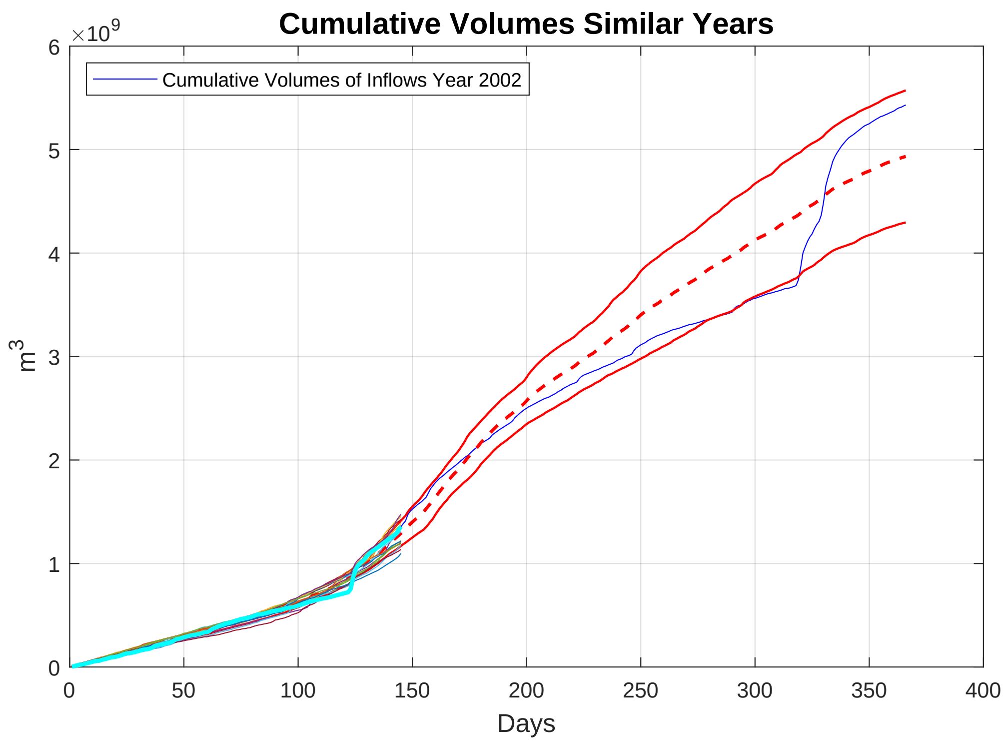


Figure 6.19: Example 3: "Red band" given by the cumulative volumes of inflows of similar years and year 2002 cumulative volumes of inflows

6.1.2 "A PRIORI" PROBABILISTIC MODEL

The second approach starts from the statistical analysis of the historical data, in particular on the cumulative frequency curves of the probability of exceedance of a given volume from inflows in a given day.

For a discrete or continuous random variable, the *Cumulative Distribution Function* (CDF) denoted by $F_X(x)$, is the probability of nonexceedance of x, this is sometimes referred to as the distribution function.

$$F_X(x) = Pr[X \leq x] \tag{6.4}$$

$F_X(x)$ is a monotonic function, which, by definition, increases for increasing values of X and for all possible values of x:

$$0 \leq F_X(x) \leq 1 \tag{6.5}$$

In the case of a discrete random variable, $F_X(x)$ is the sum of the probabilities of all possible values of X that are less than or equal to the argument x. That is:

$$F_X(x) = \sum_{X_k \leq x} p_X(x_k) \tag{6.6}$$

This is summed over all possible X_k less than or equal to x [Kottegoda and Rosso, 2008].

For our purpose is useful to ask how often the random variable is above a particular level. This is called the *Complementary Cumulative Distribution Function* (CCDF) or simply the *Tail Distribution* or exceedance, and is defined as:

$$\bar{F}_X(x) = Pr(X > x) = 1 - F_X(x) \tag{6.7}$$

Like for the Empirical Method, it's chosen a starting day for the forecasts, from which we want to plan the lake's regulation. In this case it isn't done directly a comparison with the historical data series, but from them it's done a CCDF of the cumulative volumes of inflows for each day in the year.

Since the regulation will be based on the forecasts of future inflows volumes, it's studied the statistics of the cumulative volumes 15 days over the starting day. Are chosen 15 days because they correspond to a period of two weeks for the regulation policy, as it to say "short term" regulation.

The historical series comprehend 74 years of observations, so in each day it's analyzed a sample of 74 cumulative volumes. The greater value will have a probability of exceedance of $\frac{1}{74+1} = 0.01334$ (Weibull Formula for sample distribution), the lowest one will have a probability of exceedance of $\frac{74}{74+1} = 0.9866$.

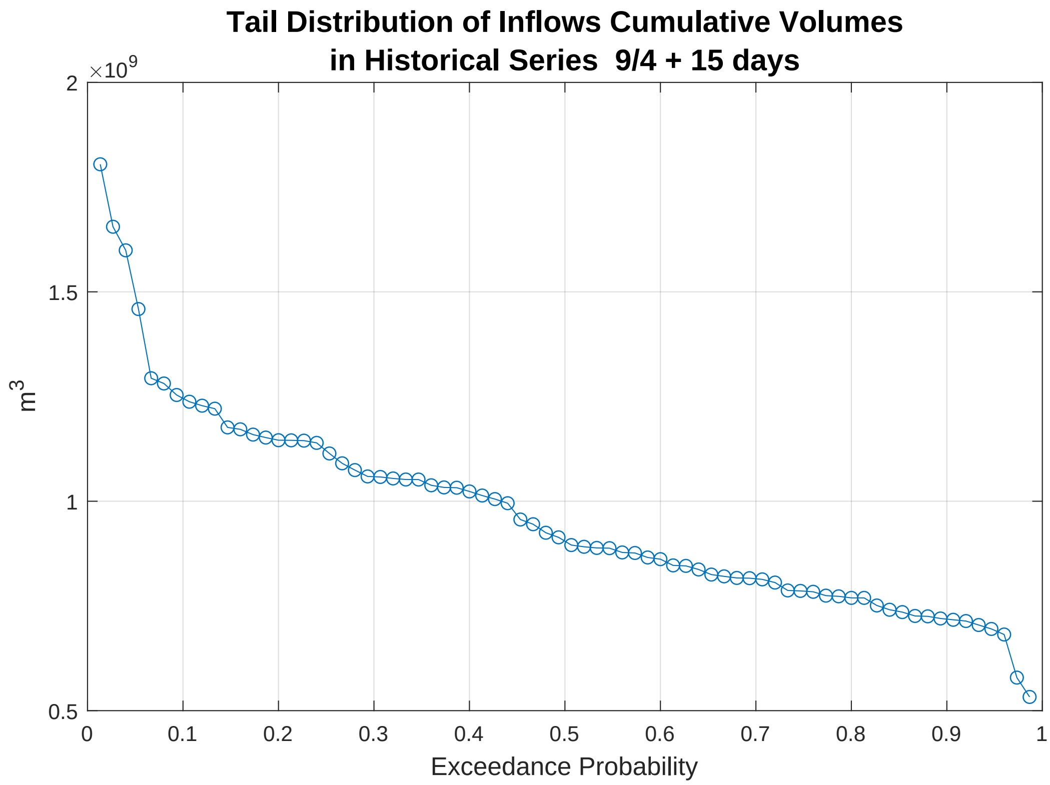


Figure 6.20: Tail distribution of inflows cumulative volumes from historical data series on day 24/04, that is 15 days over the starting day 09/04

Some significant levels of exceedance probability are fixed, corresponding to specific return period values.

- $\bar{F}_X(x) = Pr(X > x) = 95\%$ —-> $T_R = \frac{1}{0.95} = 1.05$ years
- $\bar{F}_X(x) = Pr(X > x) = 80\%$ —-> $T_R = \frac{1}{0.80} = 1.25$ years
- $\bar{F}_X(x) = Pr(X > x) = 50\%$ —-> $T_R = \frac{1}{0.50} = 2$ years
- $\bar{F}_X(x) = Pr(X > x) = 5\%$ —-> $T_R = \frac{1}{0.05} = 20$ years

From the tail distribution, in Fig.6.20, are extracted the threshold values of inflows cumulative volumes for the levels of probability previously described. Clearly, for each day of the year these threshold values will change since they are deducted from different exceedance distributions.

The threshold cumulative volumes found are independent from the ones of reference, for this reason the method it's called probabilistic "a priori" since it's only based on the historical data.

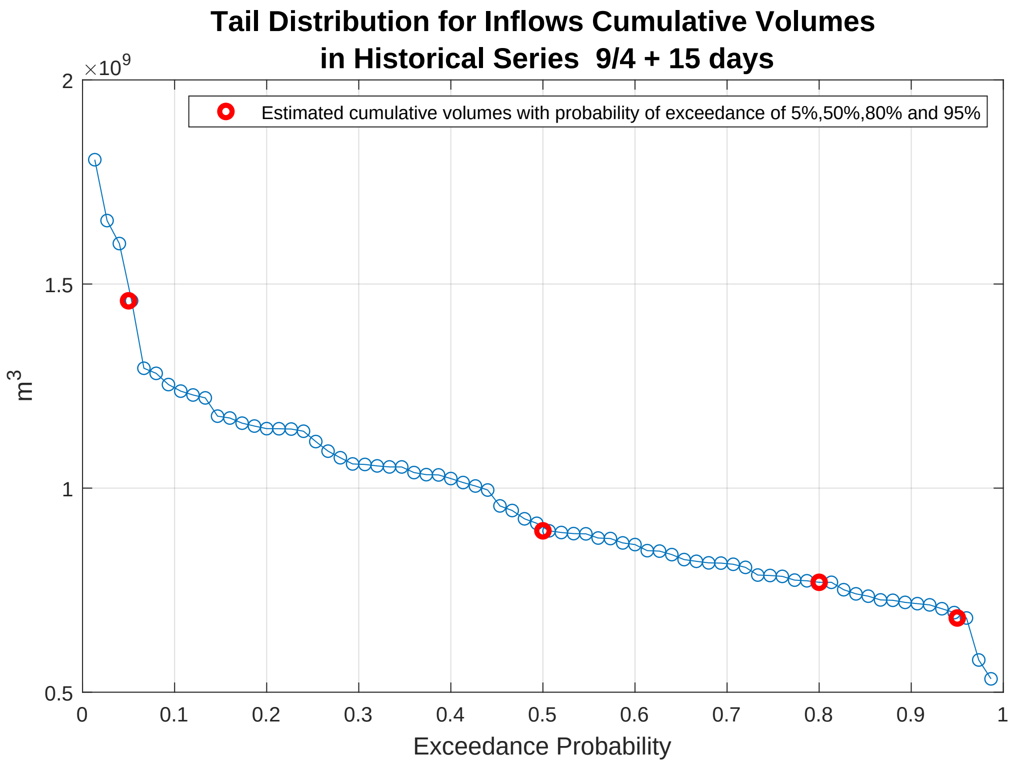


Figure 6.21: Tail distribution of inflows cumulative volumes on day 24/04 and estimated threshold volumes for different levels of exceedance probability

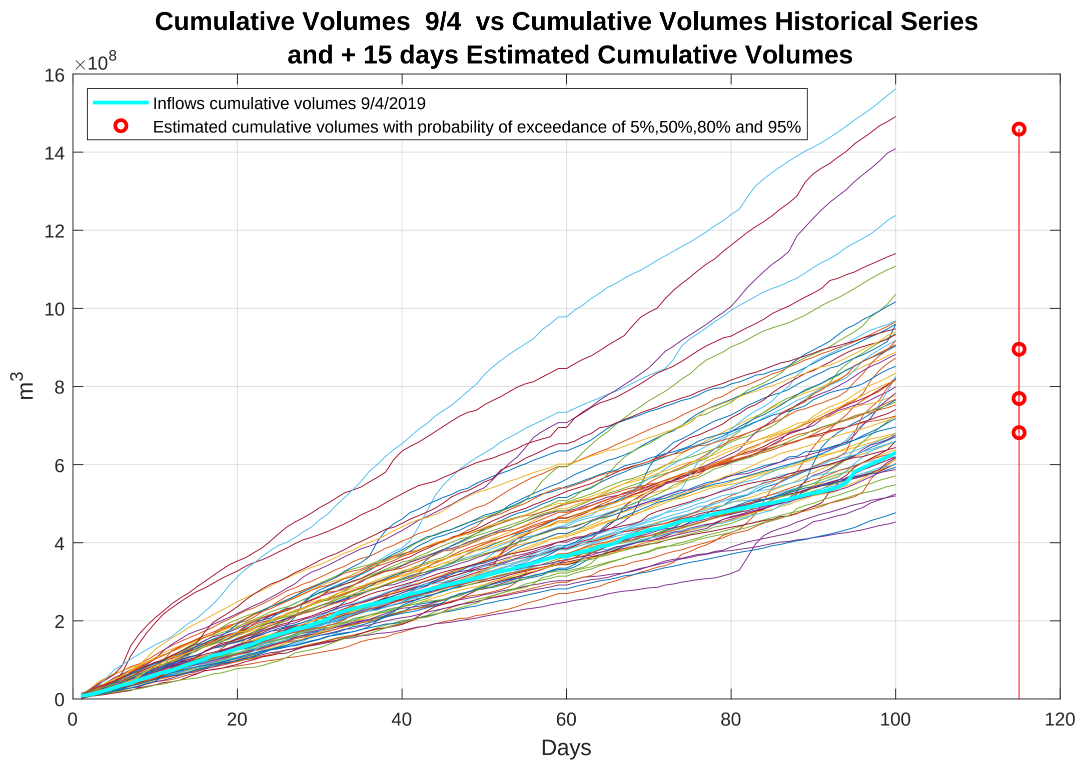


Figure 6.22: Cumulative volumes from historical series till 09/04 and estimated threshold volumes for different levels of exceedance probability on 24/04

CHECK OF THE "A PRIORI" PROBABILISTIC MODEL

Differently from the Emprical Model, for any starting day, the forecasts of the inflows volumes lasts only 15 days. The regulation will be planned just for this period of time and then has to be re-planned, this is a big limit since it's not possible to do a long term regulation policy.

Anyway, since the forecasts are "a priori" and independent from the reference series, it's possible to connect one after the other the estimated threshold volumes for interval of 15 days and see what could be the trend in the long term.

Like for the other method, the validity is tested many times by plotting the effective cumulative volumes of inflows for the whole year and this time and by comparing them with the red points of the estimated threshold volumes.

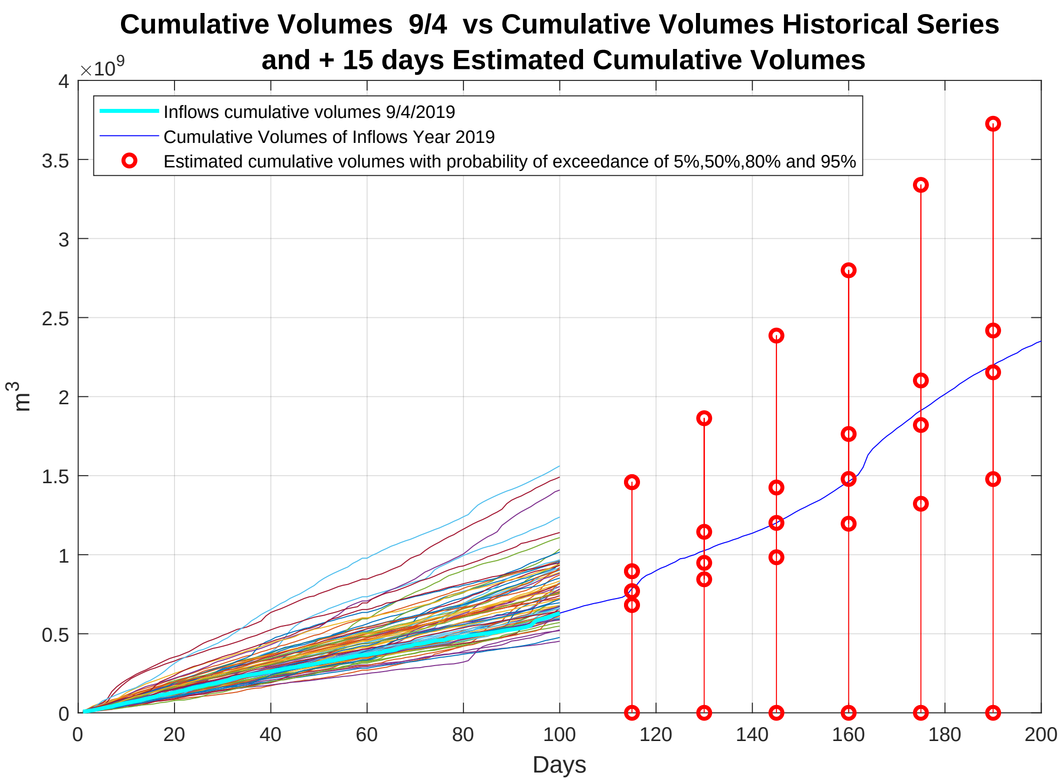


Figure 6.23: Cumulative volumes from historical series till 09/04/2019 and estimated threshold volumes for different levels of exceedance probability every 15 days

From the results it can be noticed that year 2019 is a quite dry year, so in general the line of cumulative volumes of inflows stays near the probability of exceedance of 80%. The same reasoning is for Example 2, the year is rainy compared to the mean of the other years, and the curve stays between the exceedance of 5% and 50%.

The trouble is that the model refers always to the historical series, without paying any attention on observed data in the reference year. The "red dots" will be the same, no matter how has gone the rest of the year. It's role of the decision maker to correctly understand the previsions and plan the regulation. The estimated volumes give an idea of the quantity of water that with a certain probability will go in the reservoir.

Example 1: Starting date of the forecasts 09/08/1966

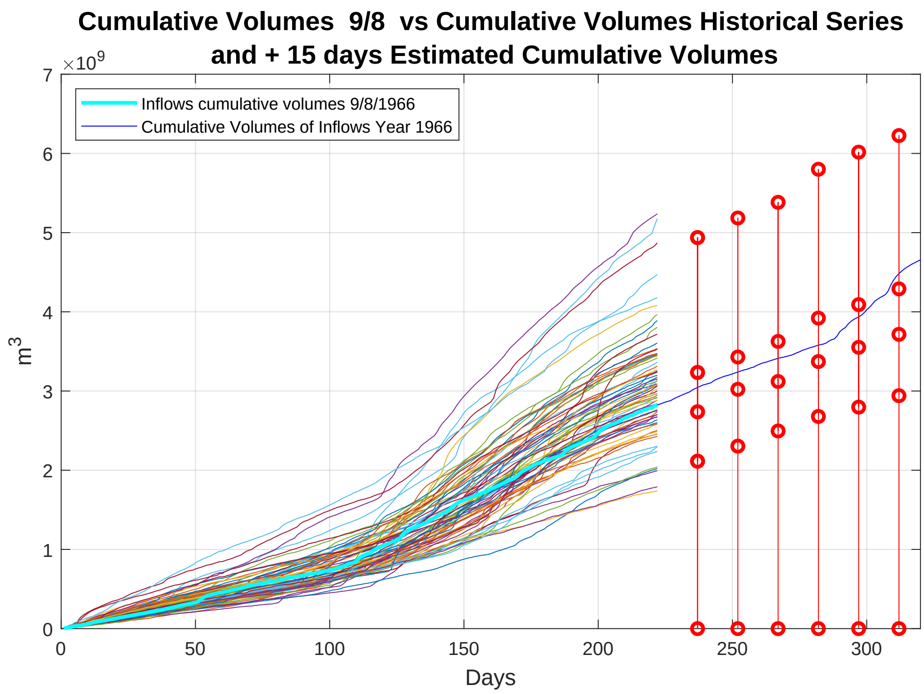


Figure 6.24: Cumulative volumes from historical series till 09/08/1966 and estimated threshold volumes for different levels of exceedance probability every 15 days

Example 2: Starting date of the forecasts 25/06/1971

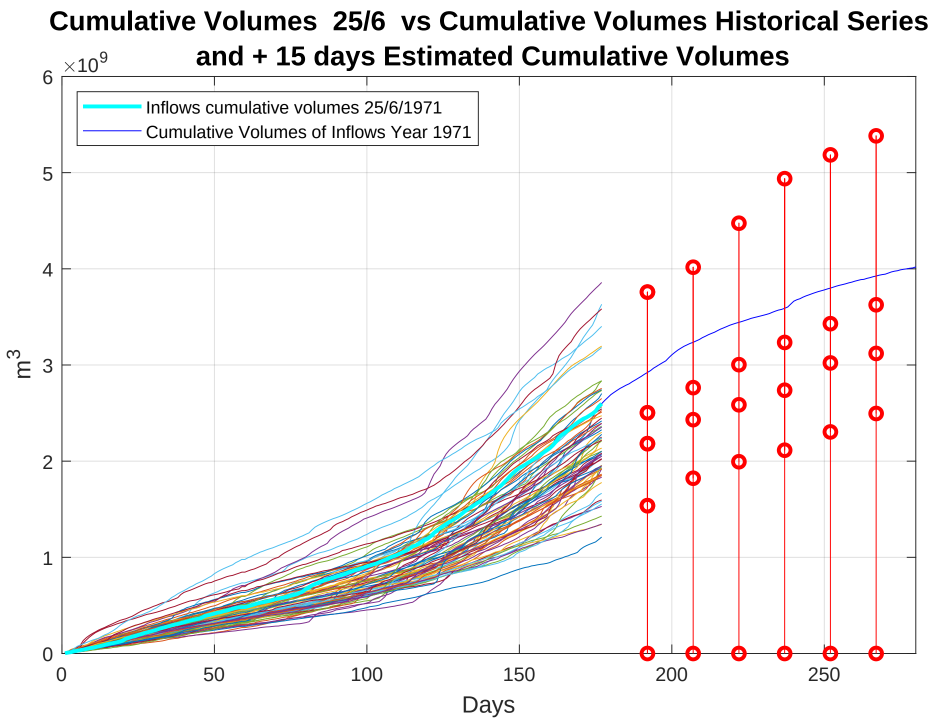


Figure 6.25: Cumulative volumes from historical series till 25/06/1971 and estimated threshold volumes for different levels of exceedance probability every 15 days

6.1.3 "A Posteriori" Probabilistic Model

The last approach begins from the main limit of "a priori" probabilistic model, or rather that the model refers always to the historical series without paying any attention on observed data. This aspect needs to be improved especially for exceptionally wet or dry years. For instance year 2001, particularly rainy, isn't suitable for "a priori" method: inflows cumulative volume value on 25/06/2001 is already higher than the estimated cumulative volumes for 15 days later, it's quite impossible that the prevision will be right.

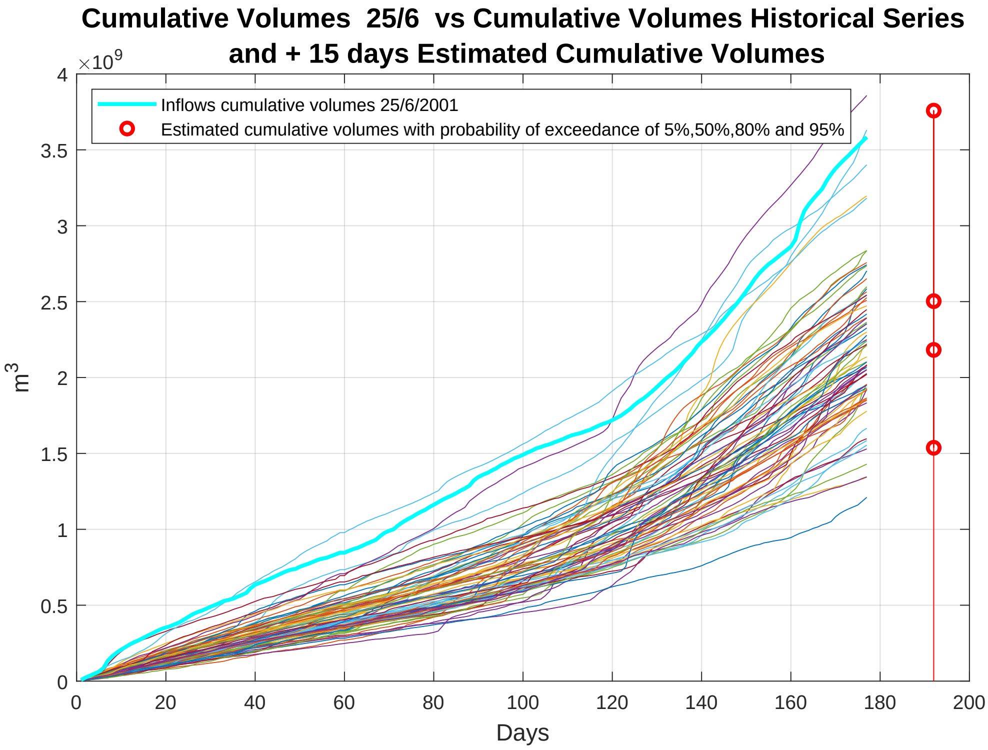


Figure 6.26: Application of "a priori" probabilistic model in year 2001

The way for incorporating observed data to probability method, is *Bayes's Theorem* and conditional probability. In probability theory and statistics, Bayes's theorem estimates the conditional probability of a given state of a population after a sample has been observed; by updating the prior probabilities, the engineer can assess the likelihood of design events by incorporating the additional information given by conditioned posterior probabilities [Marrero, 2016].

In the discrete case, let $A_1,...,A_n$ be a set-theoretic partition of the sample space, with the probability $P(A_i) > 0$ for $i = 1,...,n$. If B is another event with $P(B) > 0$, then:

$$P(A_k|B) = \frac{P(B|A_k)P(A_k)}{\sum_{i=1}^{n} P(B|A_i)P(A_i)} \qquad k = 1,...,n \tag{6.8}$$

The proof follows from the expressions for conditional probabilities

$$P(A_k|B) = \frac{P(A_k \cap B)}{P(B)} \quad and \quad P(B|A_k) = \frac{P(B \cap A_k)}{P(A_k)} \tag{6.9}$$

and the law of total probability

$$P(B) = \sum_{i=1}^{n} (B|A_i)P(A_i) \tag{6.10}$$

Simply talking:

$$P(A|B) = \frac{P(B|A)P(A)}{P(B)} \tag{6.11}$$

where:

- $P(A)$: prior probability, or marginal probability, it doesn't take into account any information about B, is the probability of an event before new data is collected

- $P(A|B)$: conditional probability of A, known B, namely posterior probability, since is derived from the value of B

- $P(B|A)$: conditional probability of B, known A

- $P(B)$: prior probability, it is used as normalization constant

The application of Bayes's theorem for inflows cumulative volumes forecasts regards the changing of the tail distribution from historical data series on 15 days later the starting day of forecasts t_{i+1}, basing on the cumulative volume of inflows W_i observed on that day t_i.

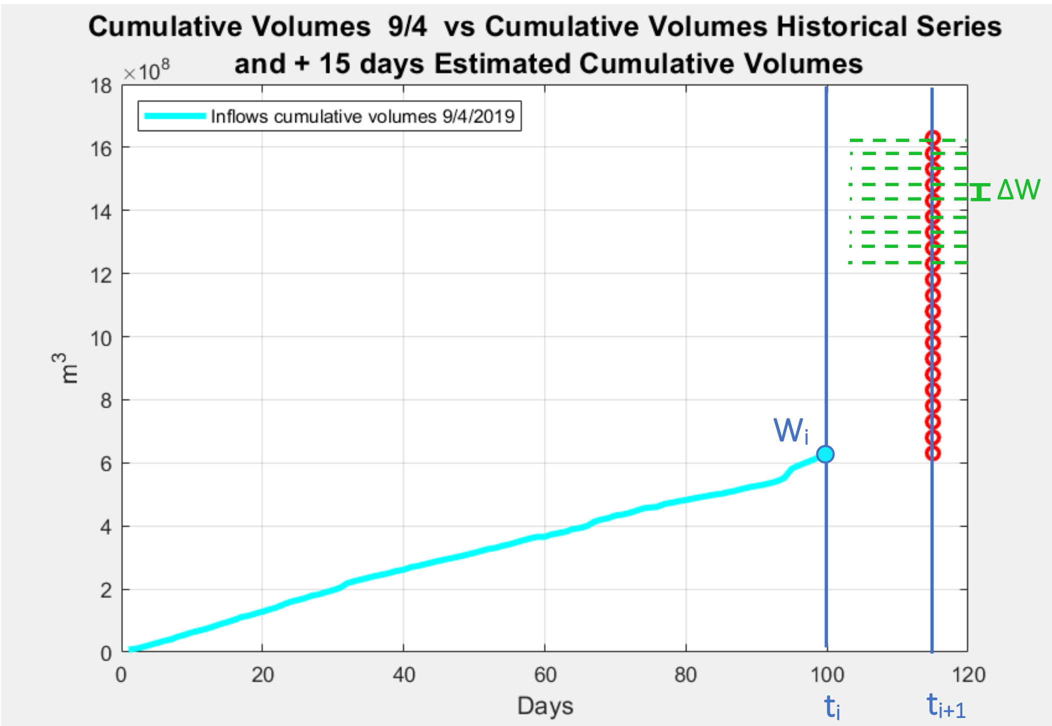


Figure 6.27: Application of "a post" probabilistic model

Cumulative volumes on day t_{i+1}, are subdivided in classes by a certain ΔW value, then will be searched the conditional probability of this values by Bayes's theorem. The increment of volume within 15 days, has to be reasonable, analysing historical data series, $\Delta W = 10^7 m^3$ is found as possible increment.

$$W_{i+1} = W_i + N\Delta W \tag{6.12}$$

With N, integer number between 0 and 100, the classes are made for $N = 5, 5, 10, 15, 20, ..., 95, 100$:

$$W_{1,i+1} = W_i + 5\Delta W = W_i + 0.5 * 10^8 \tag{6.13}$$

$$W_{n,i+1} = W_i + 100\Delta W = W_i + 10^9 \tag{6.14}$$

The Bayes's theorem in terms of cumulative volumes of inflows, result written as:

$$P(W_{i+1}|W_i) = \frac{P(W_i|W_{i+1})P(W_{i+1})}{P(W_i)} \tag{6.15}$$

where:

- $P(W_{i+1})$: prior probability, or marginal probability, it doesn't take into account the observed inflow W_i, is the probability exceedance coming from historical data series on day t_{i+1}

- $P(W_i)$: prior probability, it is used as normalization constant, is the probability exceedance coming from historical data series on day t_i

- $P(W_{i+1}|W_i)$: conditional probability of W_{i+1}, known W_i, namely posterior probability, since is derived from the observed inflow

- $P(W_i|W_{i+1})$: conditional probability of W_i, known W_{i+1}

The first step is to compute prior probability of $P(W_{i+1})$, through historical series on day 24/04:

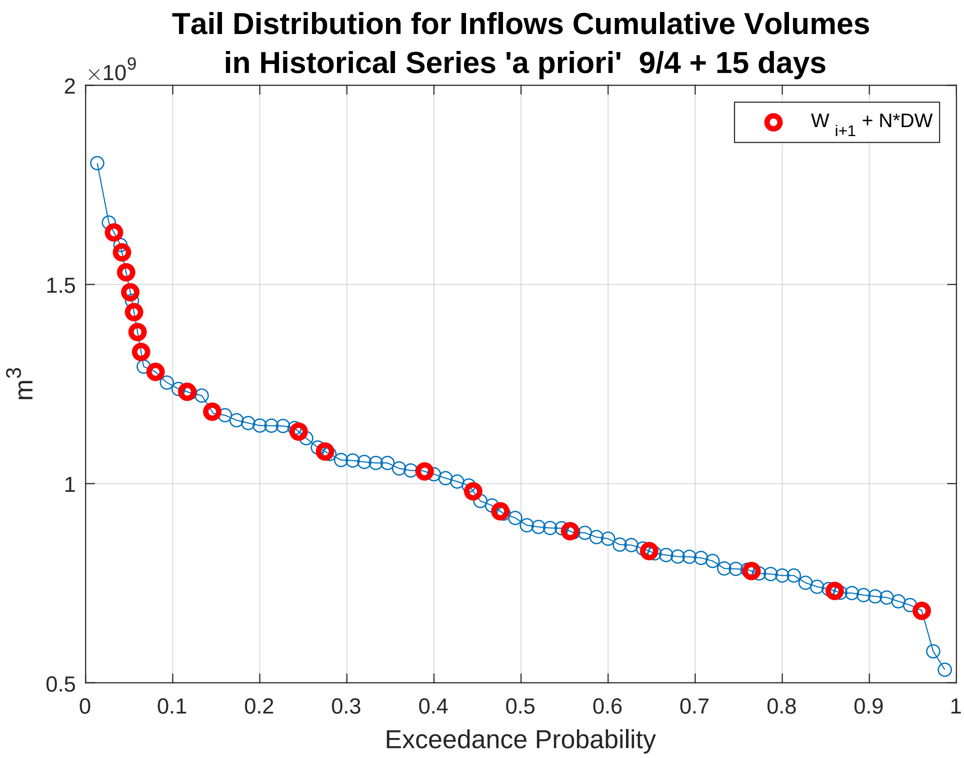


Figure 6.28: Tail distribution for inflows cumulative volumes in historical series "a priori", $P(W_{i+1})$

Then prior probability of $P(W_i)$ is computed, taking observed inflow on 09/04 and comparing it with historical data:

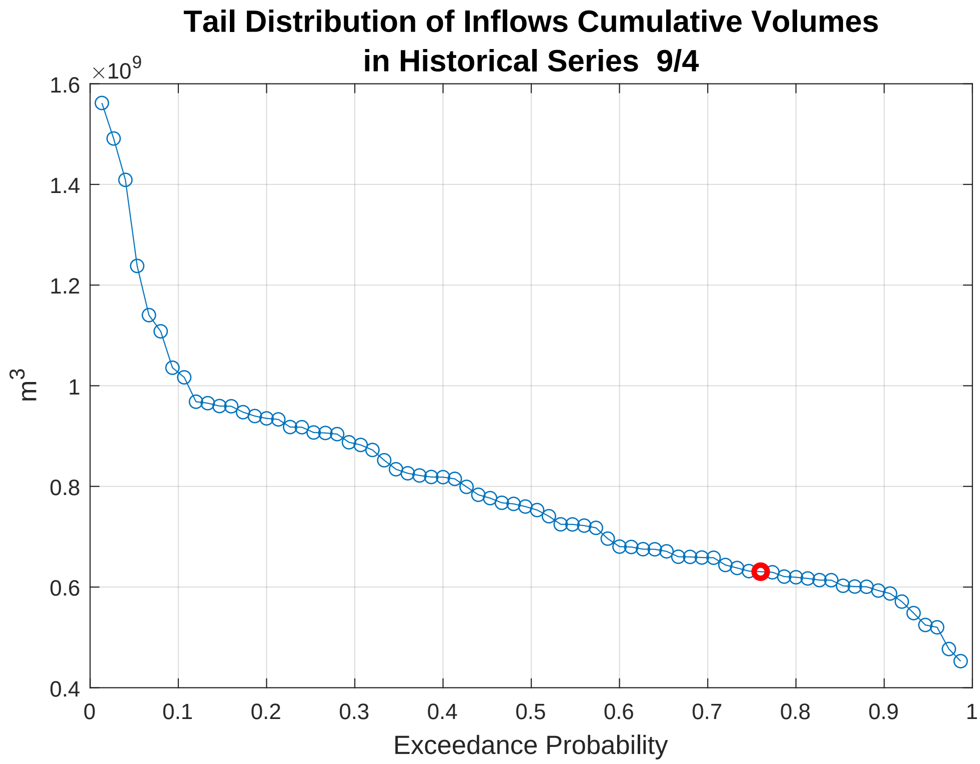


Figure 6.29: Tail distribution for inflows cumulative volumes in historical series "a priori", $P(W_i)$

The computation of $P(W_i|W_{i+1})$ is more complex, from the historical series are computed differences between t_{i+1} and t_i cumulative volumes

$$W_{i+1} - W_i = w_i \tag{6.16}$$

The conditional probability of W_i, known W_{i+1}, will correspond to the exceedance probability of each increment of volume from tail distribution of differences w_i.

Being that volume increments series is the same for all the days of the year and for all the years, it can happen that some $N\Delta W$ values stays outside the tail distribution values.

When this happens because w_i values are too big, it's assumed $P(W_{i+1}|W_i) = 0$ on the opposite case, when increments are outside the distribution values because are too small, it's assumed an exceedance probability of $P(W_i|W_{i+1}) = 0.998$.

Finally it's computed the conditional probability of future inflows cumulative volumes W_{i+1}, known W_i, through Bayes's formula: $P(W_{i+1}|W_i)$.

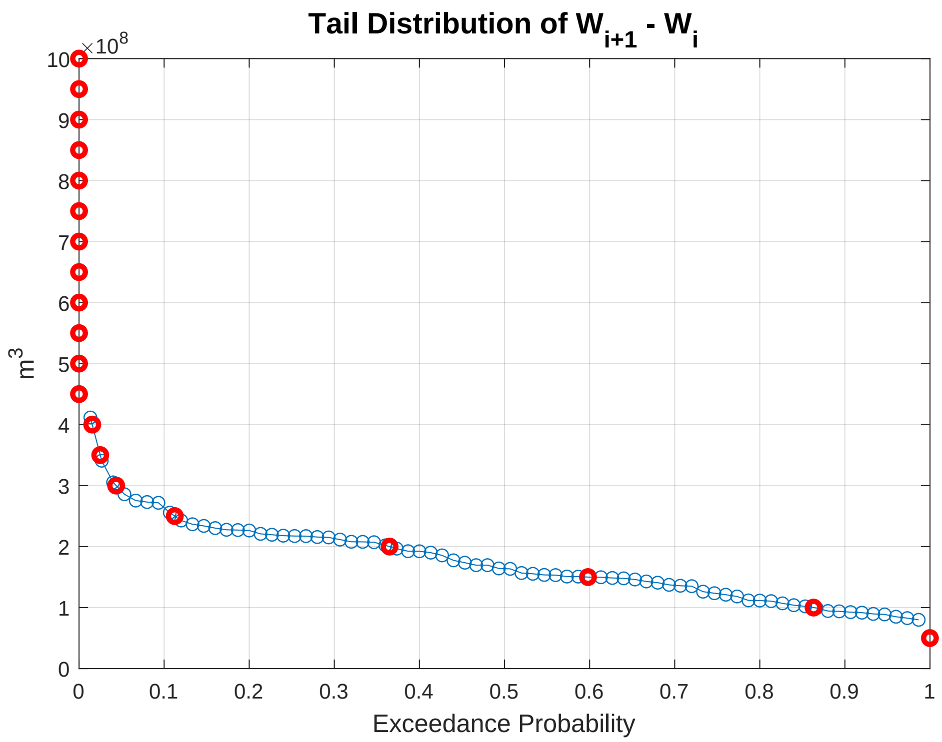


Figure 6.30: Tail distribution for difference of inflows cumulative volumes in historical series, $P(W_i|W_{i+1})$

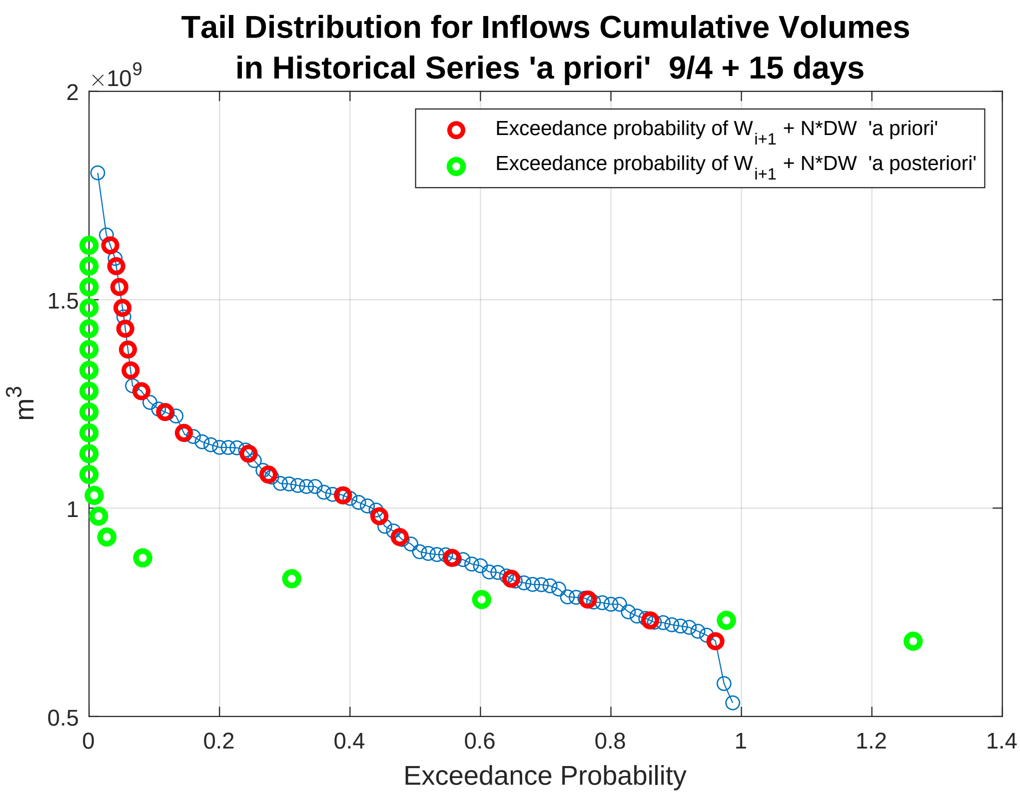


Figure 6.31: Tail distribution for inflows cumulative volumes $P(W_{i+1})$ and $P(W_{i+1}|W_i)$

The shape of tail distribution for conditional probability is very different from the one of starting.

Many values result to have an exceedance probability very close to zero, related to increments of volume on the left end in Fig. 6.34; other values have a probability of exceedance bigger than 1, this value is actually impossible.

The error in the computation can be caused by too low (or high) values of cumulative volumes, linked to a probability very close to 1 (or to 0), the same for small volume increments, another cause can simply be the relationship among the three elements in the formula that create an unbalance, however further analysis should be done for a better understanding of errors.

Taking the example of before, inflows projections starting from day 25/06/2001 and keeping the same series for W_{i+1} with the same values for increments, results:

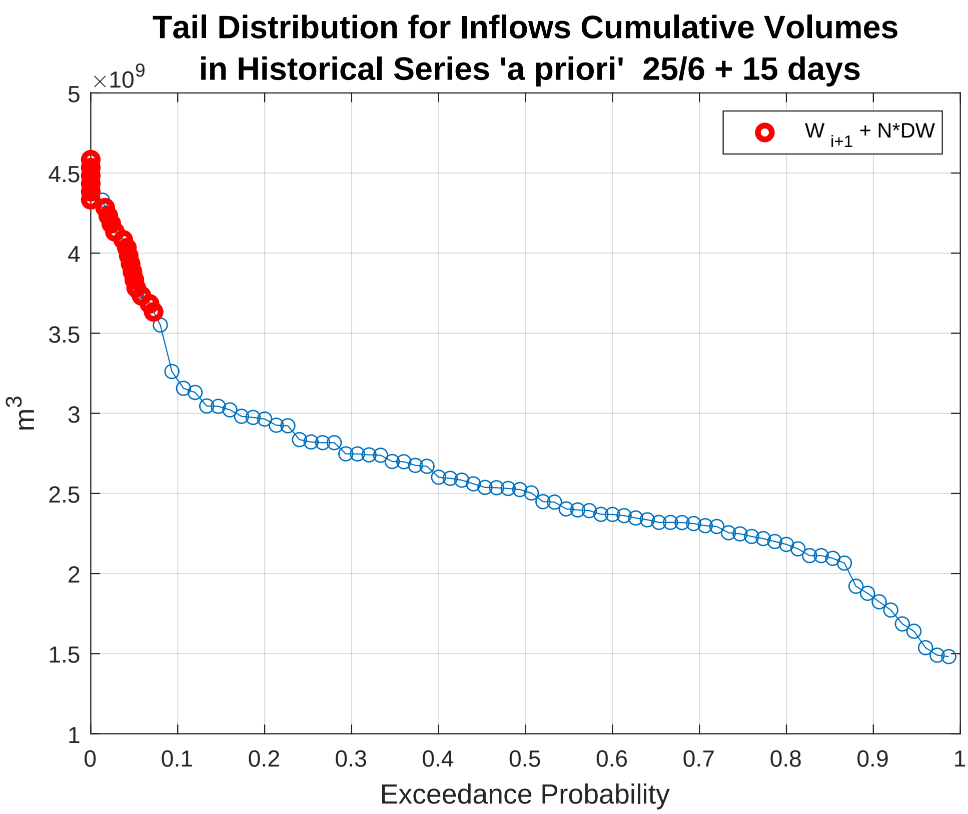


Figure 6.32: Example: Tail distribution for inflows cumulative volumes differences , $P(W_{i+1})$

All the "red dots" stay on the very high part of the curve, this because the cumulative volume W_i on 25/06 is already high, while increments are the same of before and lead to even higher values for W_{i+1}.

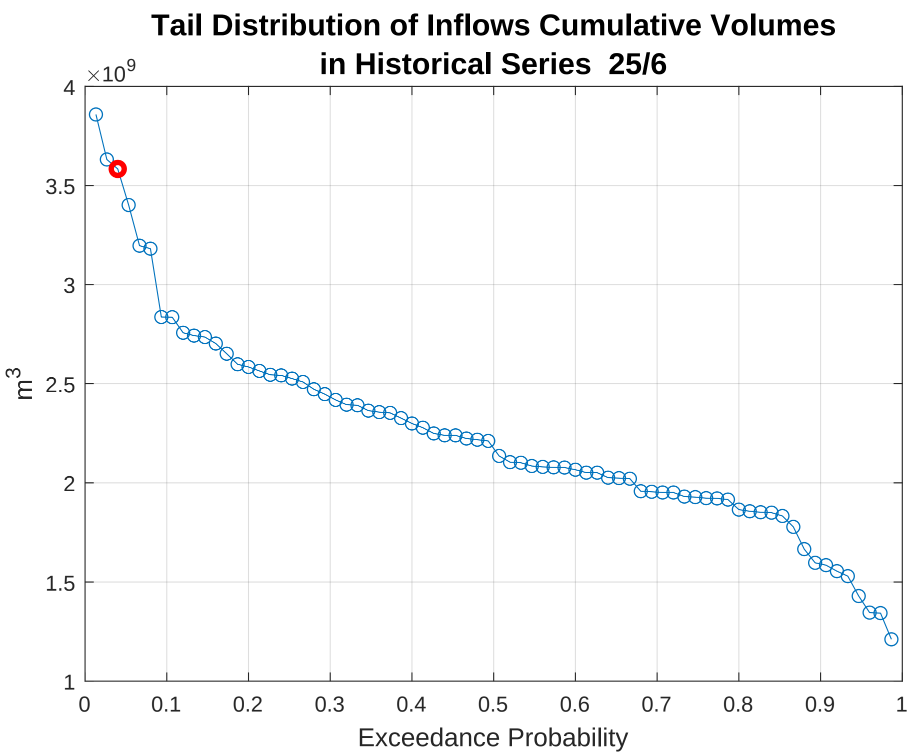


Figure 6.33: Example: Tail distribution for inflows cumulative volumes in historical series "a priori", $P(W_i)$

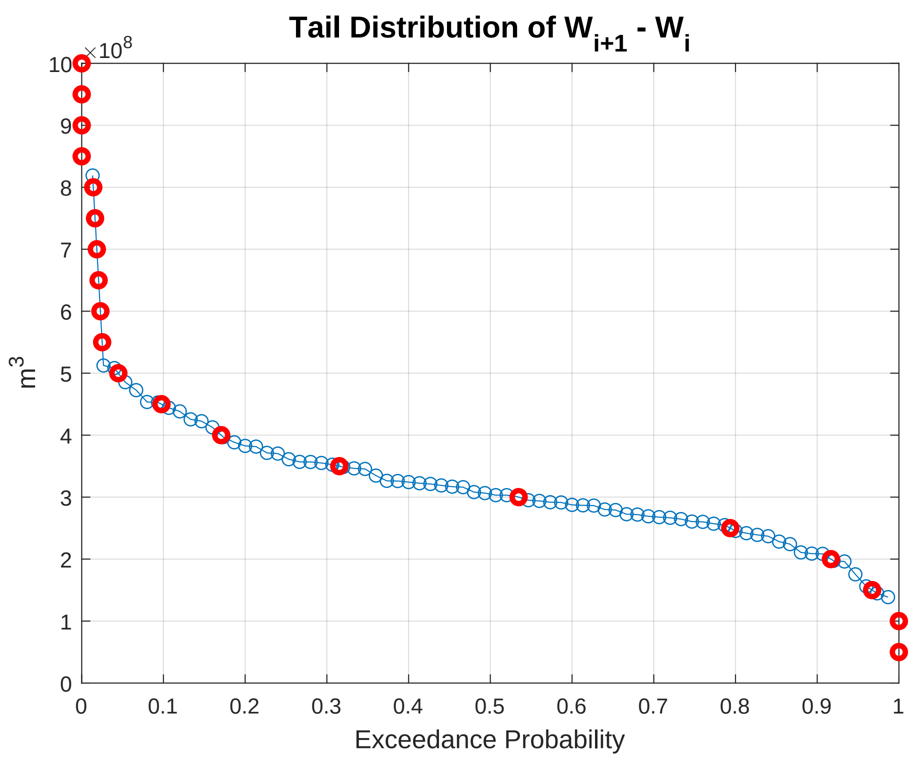


Figure 6.34: Example: Tail distribution for difference of inflows cumulative volumes in historical series, $P(W_i|W_{i+1})$

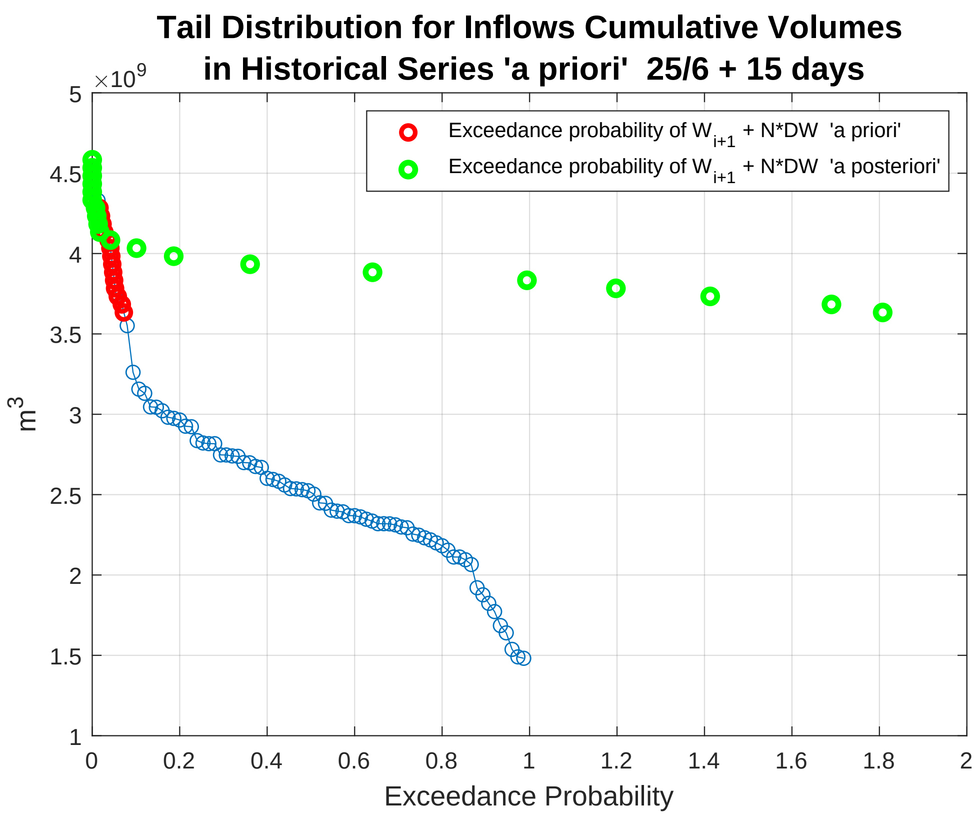


Figure 6.35: Example: Tail distribution for inflows cumulative volumes $P(W_{i+1})$ and $P(W_{i+1}|W_i)$

6.1.4 Comparison among the three forecast models

All the three methods present pros and cons:

- Empirical model makes use of the most similar years for the computation of future inflows, the "red band" works quite well, except for sudden events when a huge amount of water goes into the lake in few days. For extremely wet or dry years the empirical approach behaves correctly, considering historical critical years and widening the "red band". In respect of this, the number of similar years is reduced to 10, to keep from taking extreme years, that enlarge the band even when it's not necessary.

- "A priori" probabilistic model major limit is that doesn't contain any reference to the observed data, that is the current year. It works well once known the inflow curve and observing its position compared to the "red dots" of estimated cumulative volumes, it gives an idea of characteristics of current year.

- "A posteriori" probabilistic model is similar to the "a priori" one. The aim is to correct prior probability basing on observed data, especially in the case of extreme years. The major problem is that it doesn't work well for this type of years, creating impossible "a posteriori" tail distributions.

Bayes' theorem is particularly useful for experiments carried out in stages. This has a potentially important role in engineering applications because it provides a method for continuously incorporating new information with previous data. By updating the prior probabilities, the

engineer can assess the likelihood of design events by incorporating the additional information given by conditioned posterior probabilities [Kottegoda and Rosso, 2008].
Further developments should be done for the application of Bayesian method to this work, since it requires calibration and a larger number of samples, here only 74 for each daily tail distribution. The application of Bayes also through software is a current topic, in particular for what concerns the study of precipitation and hydrology. In the study proposed by [Zhang et al., 2016] four Bayesian maximum entropy (BME) models were compared to estimate the spatial distribution of mean annual precipitation of the selected areas. Meteorological data from 48 meteorological stations were used, and spatial correlations between three meteorological factors and two topological factors were analyzed to improve the mapping results including annual precipitation, average temperature, average water vapor pressure, elevation, and distance to coastline. Some missing annual precipitation data were estimated based on their historical probability distribution and were assimilated as soft data in the BME method. The results showed that multivariate BME with soft data outperformed the other methods, indicating that adding the spatial correlations between multivariate factors and soft data can help improve the estimation performance. Despite this great result their study has limitations that should be addressed: the versatility of BME approach has yet to be verified, the use of more field-based meteorological data could improve the accuracy of spatial estimation and further developments should be done for extending the BME model from the current spatial dimensions to space-time dimensions.

Since the last method presents many limits and still unknown aspects about its applicability, the choice is to combine the first two models in an unique method.
The planning of optimal regulation policy will be done mainly having as guidelines the "red band" given by empirical model, with a number of similar years reduced to 10, while the "red dots" by probabilistic one will furnish supplementary information about the characteristics of the year, compared to all historical series, particularly useful for long term planning.

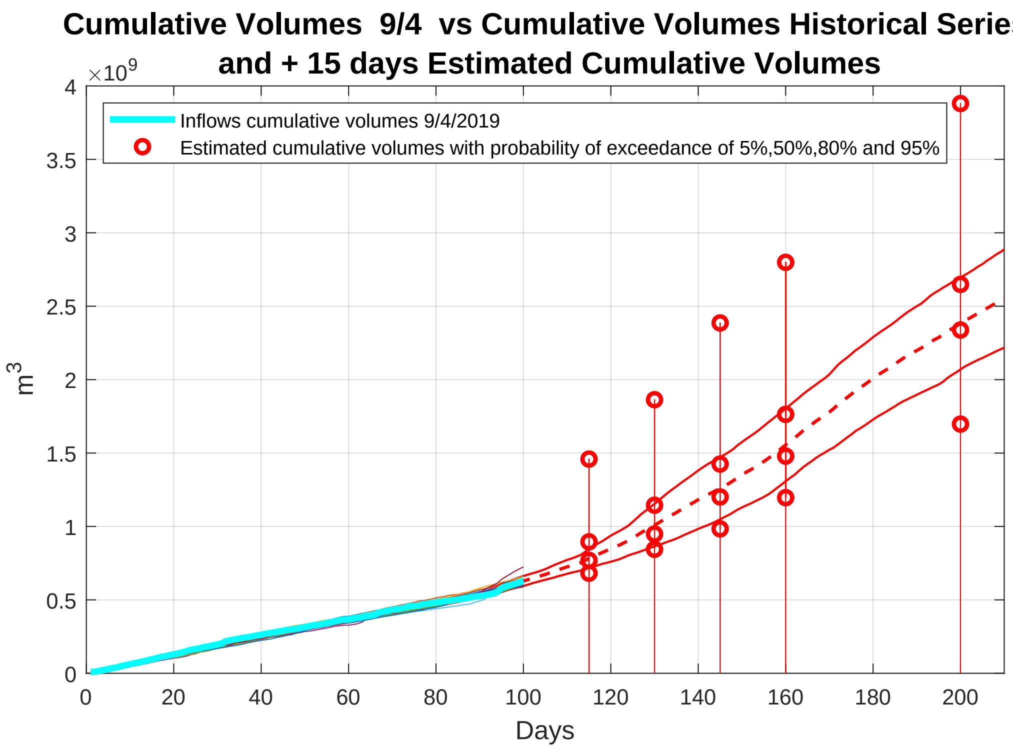


Figure 6.36: Forecasts of cumulative volumes for inflows, Method 1+2

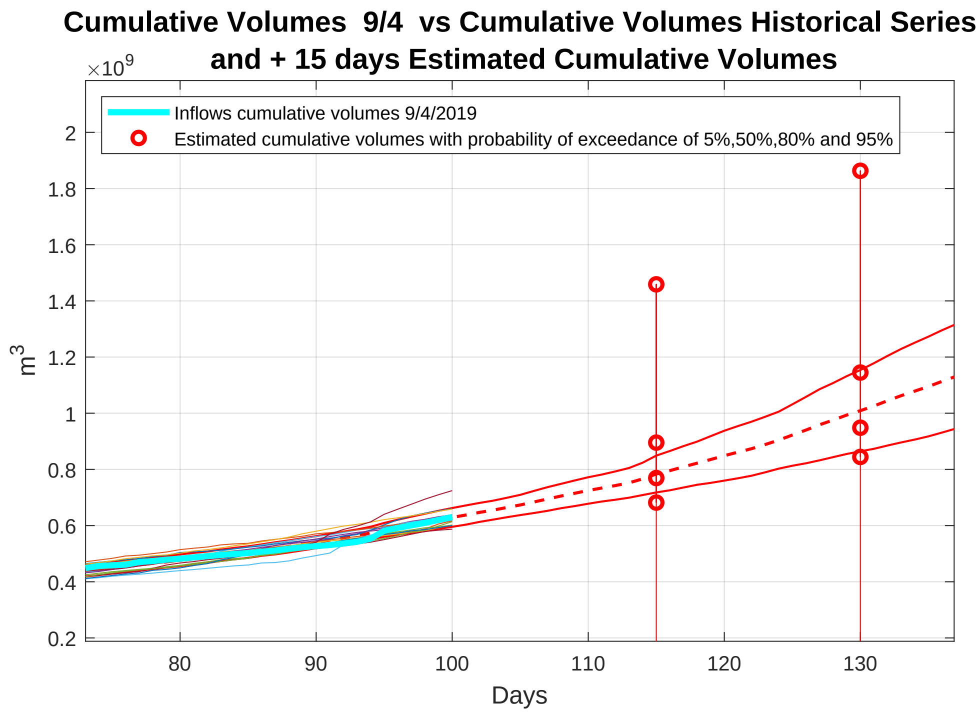


Figure 6.37: Forecasts of cumulative volumes for inflows, Method 1+2, zoom

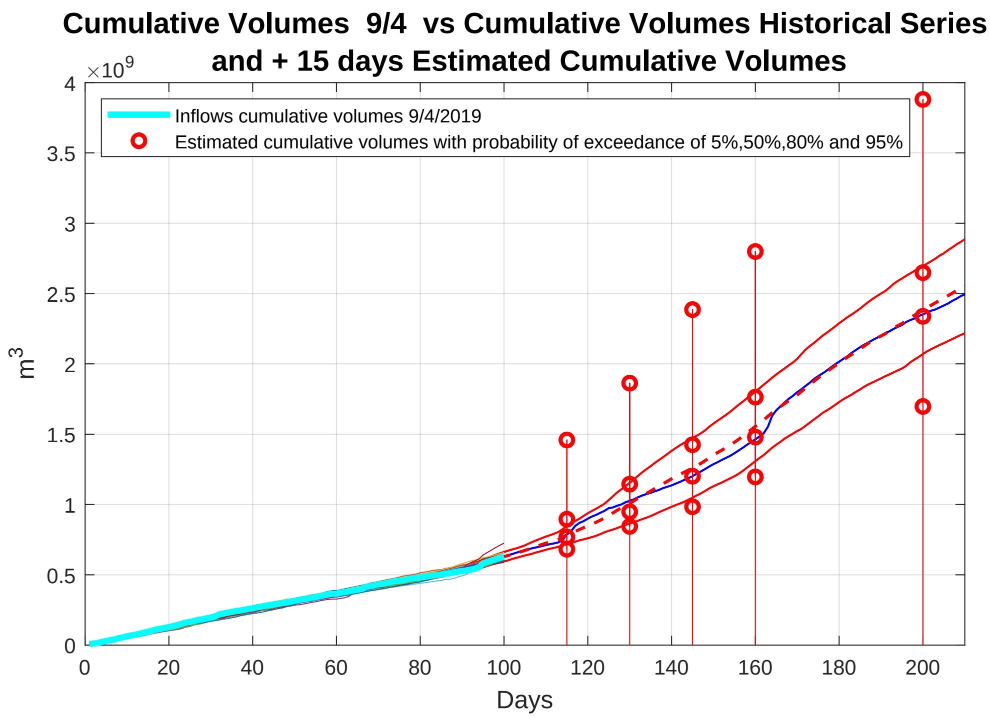


Figure 6.38: Forecasts of cumulative volumes for inflows, Method 1+2, checking validity

Note: for a better understanding in the next two chapters graphics with future projections won't report the "red dots" from the probabilistic method, while actually using the tool developed for lake regulation the management authority will have possibility to turn on also that additional info.

6.2 Regulation policy

Once chosen the forecast model, the next step is to decide the regulation policy to be applied. What makes the management authority is to set a certain opening for dam's sluice gates, associated to an outflow value. Delivered outflow depends on gates opening and lake level, since each maneuver has constant opening but inflows are never constant, lake level will change, changing also the outflow. As introduced in Section 1.1 about 100 maneuver are made during the year, this means that between one maneuver and the other, delivered outflow will change a bit and won't be perfectly constant. Below a screen of website *www.laghi.net* showing delivered outflow record for year 2019: the blue line refers to year 2019, the red one to the maximum values recorded and the yellow one to the minimum.

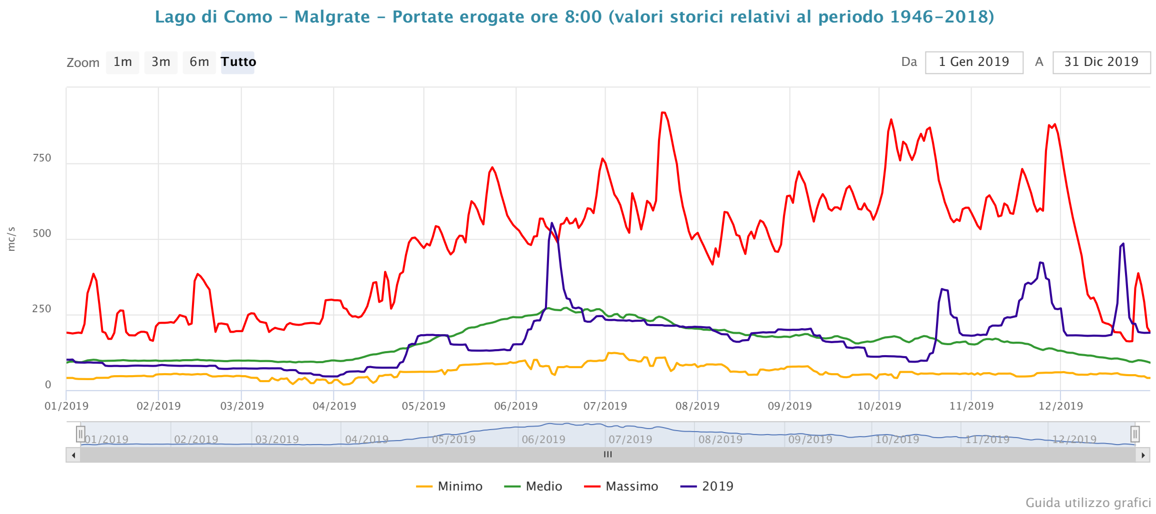


Figure 6.39: Lake Como - Malgrate - Delivered outflow at 8 a.m. [*www.laghi.net*]

All this fact is quite complex to be reproduced with precision in a computational program, for simplicity it's assumed that in each maneuver is imposed a certain outflow value, that remains constant till the next maneuver.

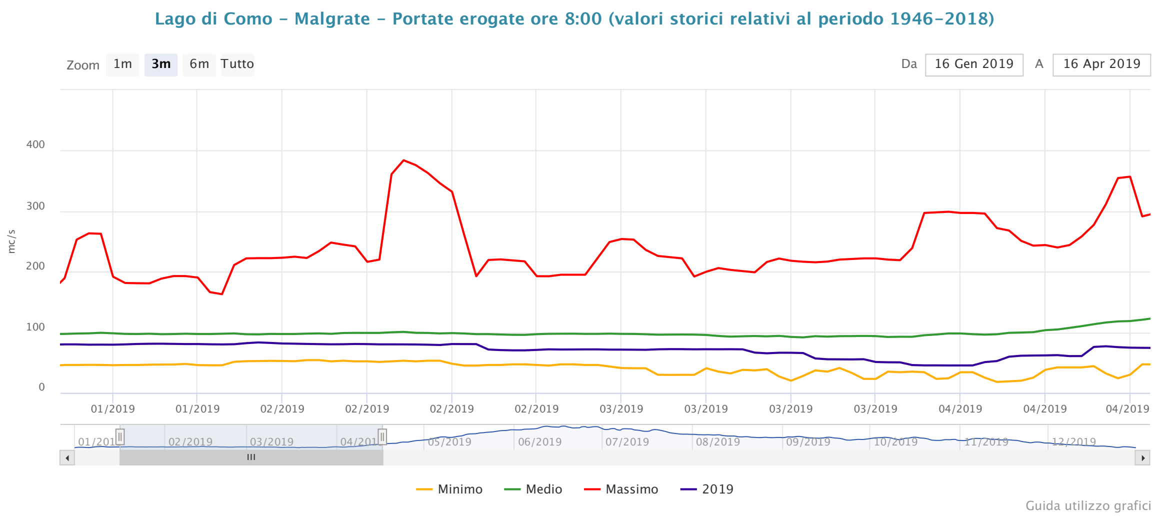


Figure 6.40: Lake Como - Malgrate - Delivered outflow at 8 a.m. [*www.laghi.net*], zoom over three months

To give an idea of the relationship between delivered outflow to be chosen and canals concessions, are computed some representative percentages of canals concessions, referring to NB curve graphic in Fig. 5.16 and reported below:

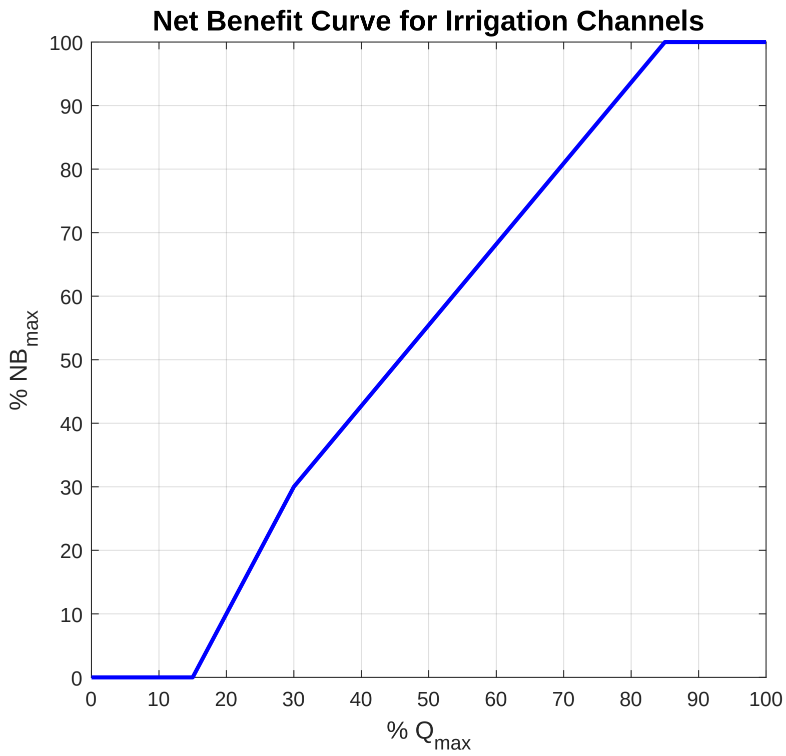


Figure 6.41: Net benefit curve for irrigation channels (in cooperation with Eng. Gandolfi)

- 100% of total canals concessions: complete fulfillment of irrigation needs

- 85% of total canals concessions: irrigation needs tot fully satisfied, but NB remains the maximum possible since crops still grow flourishing even without the optimum amount of water from irrigation

- 75% of total canals concessions: trade-off between the optimal amount of water, but too high to be always guaranteed, and a contained damage

- 50% of total canals concessions: half of canal concessions

- 30% of total canals concessions: threshold value, below this amount of water crops don't grow at all, except for precipitation supply, earnings from agriculture are almost nothing

Starting from day 09/04, that is the day of beginning of the forecasts, are plotted delivered outflows to satisfy canals concessions in different percentages.

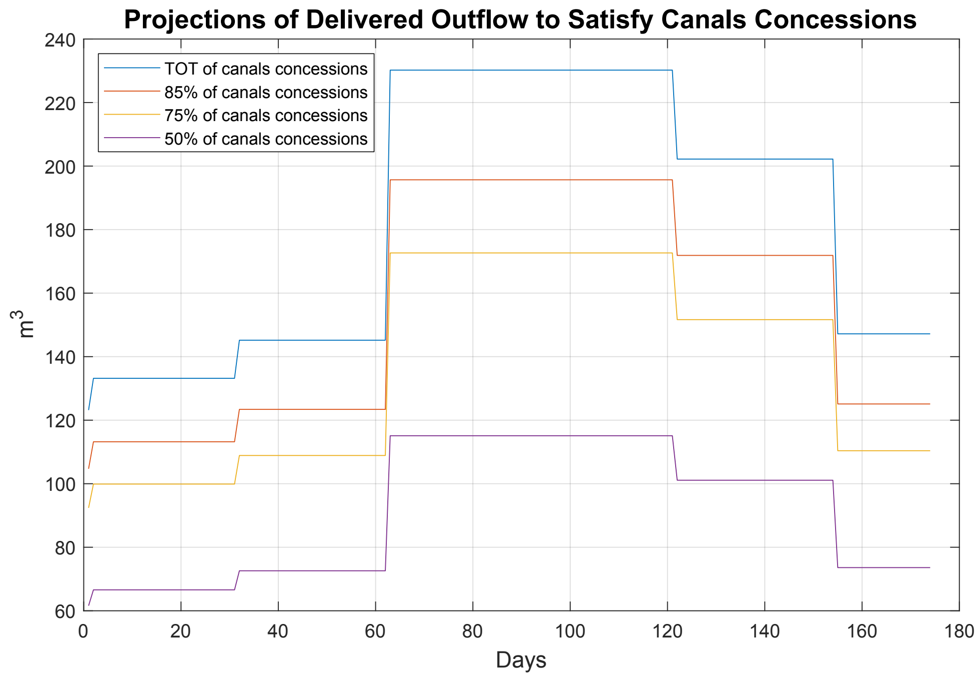


Figure 6.42: Projections of delivered outflow to satisfy canals concessions

The optimization tool works with volumes of water. Cumulative volumes associated to percentages of canals concessions are plotted starting from 09/04 and vertically translated to inflow cumulative volume value in that day, therefore that amount of water is required to be delivered to satisfy irrigation demand by the fixed percentages.

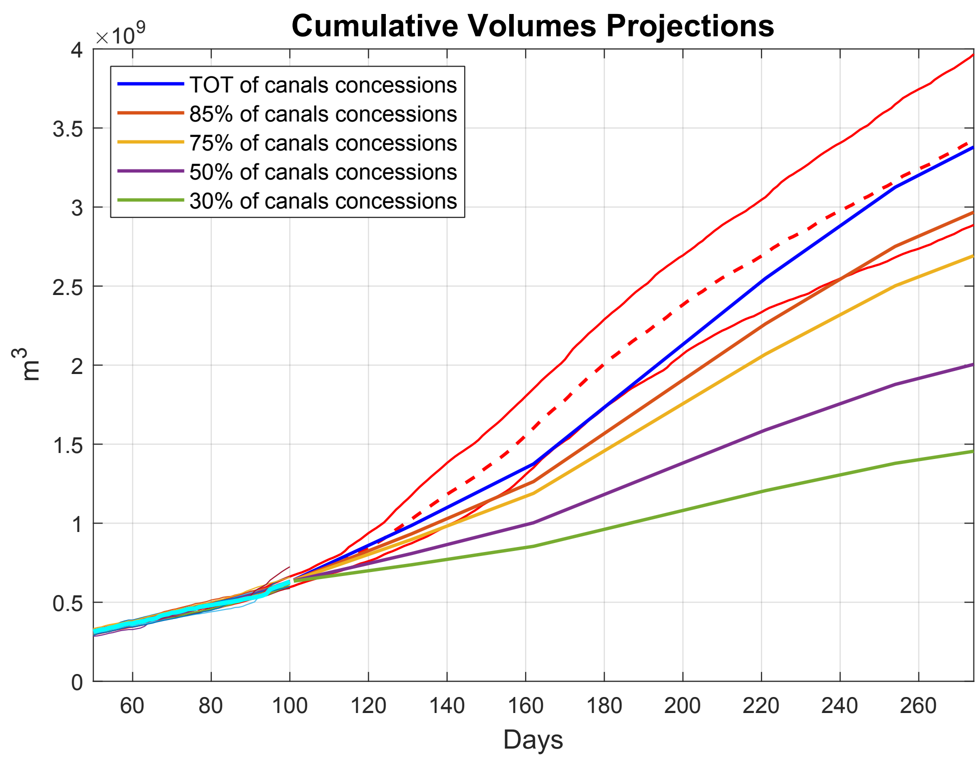


Figure 6.43: Cumulative volumes projections

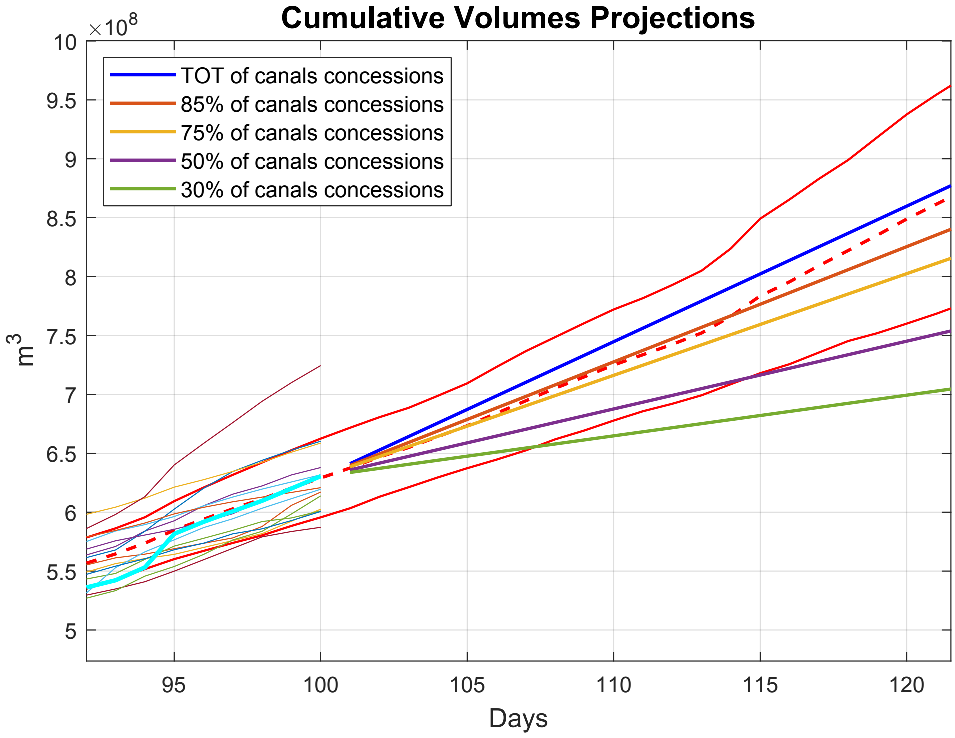


Figure 6.44: Cumulative volumes projections, zoom

As better shown in Fig. , straight lines corresponding to cumulative volumes of irrigation needs, start from the same point and have different slopes. The blue line, about complete fulfillment of irrigation needs, stays above all the other lines, since it corresponds to a bigger volume of water. What's important for the choice of regulation policy, or better for the choice of the value for delivered outflow, is to look at the relative location of concessions lines in relation to the "red band" of expected inflows.

If for example the blue line stays outside the "red band", means that future inflows probably won't be enough to sustain an outflow of such magnitude; in addiction, if the blue line stays in the top half of the "red band", means that irrigation needs could be balanced or not by inflows, depending on probability. Finally if the blue line stays below the "red band", like the green line in Fig. 6.44, means that almost surely inflows will be big enough to support canals concessions.

The last step, is to decide for how many days keep the chosen delivered outflow. This can vary basing on the period of the year in course or considering special needs. In quiet periods, like in winter, the same dam opening can be kept even for a consecutive month; on the other hand in more turbulent periods, like in autumn when flooding events occur frequently, is necessary to open and close the gates alternately for managing the flood.

A good trade-off is a period of time of 15 days, about two weeks, for this interval of time inflows forecasts are still reliable and lake level won't vary too much, in case of critical events this interval of time will be reduced to five days/one week.

The regulation tool designed asks the decision maker which delivered outflow wants to impose and for how much time, the corresponding cumulative volume is then plotted in the graphic of Fig. 6.43.

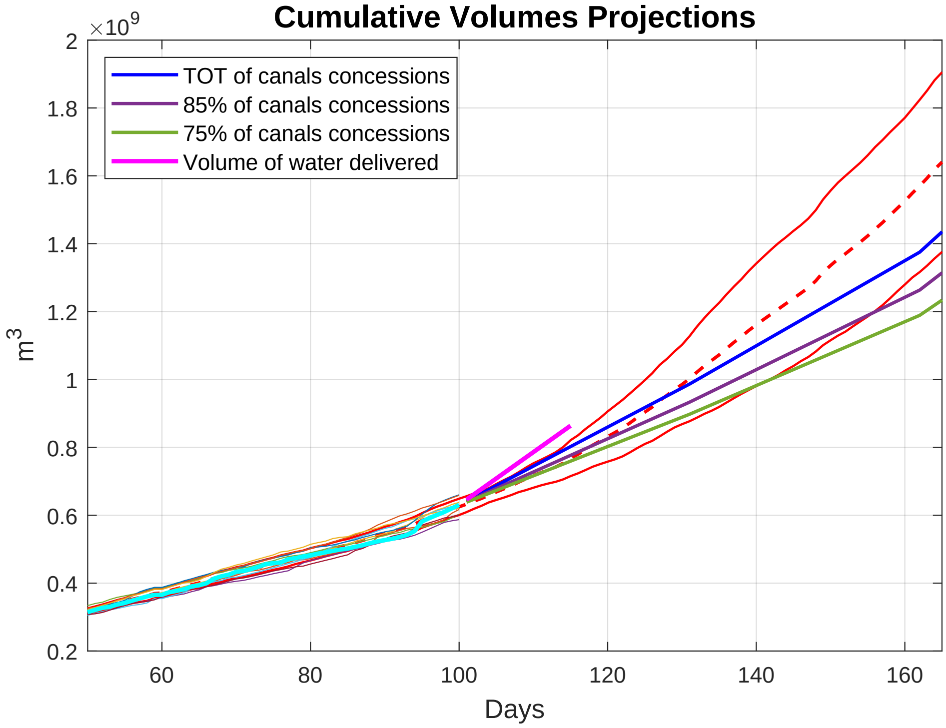


Figure 6.45: Cumulative volumes projections

The decision maker has the task of evaluating if the delivered outflow is fine or not, otherwise can propose other outflows. Below an example of three different flow rate chosen: $180m^3/s$, $135m^3/s$ and $100m^3/s$. Later in Chapter 7 will be further discussed the choosing method.

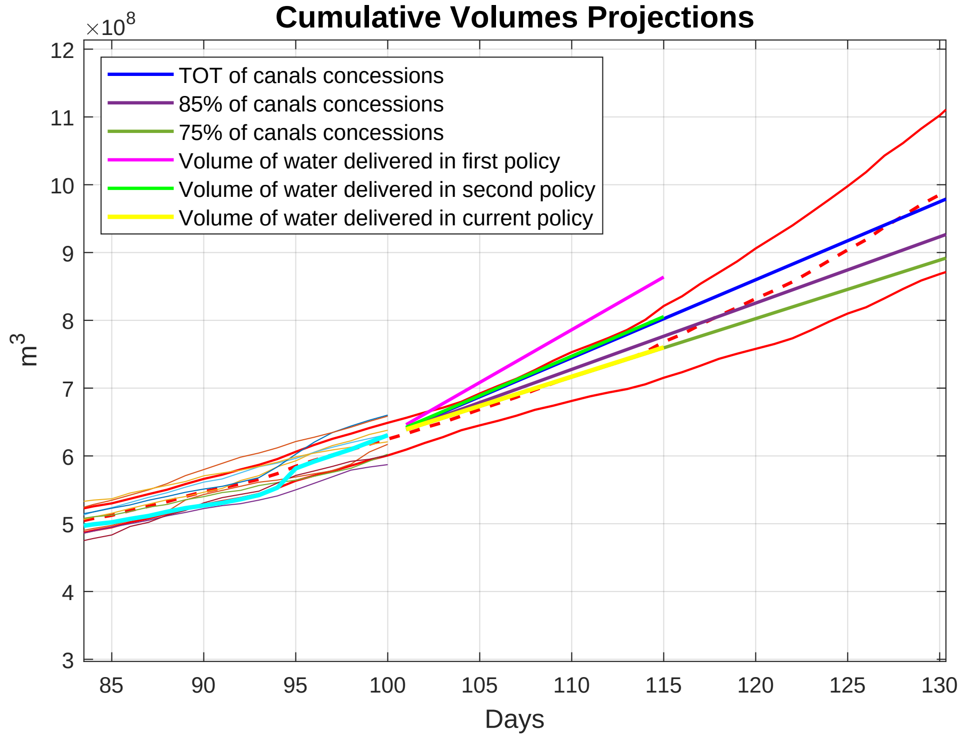


Figure 6.46: Cumulative volumes projections, three different outflows comparison

CHAPTER 7

MODEL APPLICATION

The application of the optimal regulation policy chosen is tested over the whole Adda river system: in the first part is reproduced the behavior that the lake level would have starting from the delivered outflow, in the second one the simulator computes delivered water in irrigation channels and plausible power production over downstream Adda river system, basing on the allocation policy adopted.

In order to test possible policies inflow series are taken from one fixed year from historical data. Since it isn't possible to test the method in real-time, the simulation is done assuming that the current year is one from the historical series, for example 2019, and are taken inflows and data from that year. The simulator will run as if the current year is 2019 and the decision maker is working in that time. This trick is necessary since we want to test the method all over one year. In a real application the decision maker will observe day-by-day the trend of lake level and inflows and will decide whether or not change the outflow. The program sends to the decision maker some outputs, called "warning signs", when the level of the lake reaches critical values or leaves the "optimal band".

7.1 APPLICATION FOR LAKE COMO REGULATION

The creation of a model that describes consistently reality it's not easy, there is a huge number of variables and boundaries to be respected so as to obtain a valid simulation. Many simplifications are done:

- The lake is considered to have the same surface of $145 km^2$ along all the depth, so considering all the shores to be vertical. It's an acceptable approximation since ,for the maximum gradient considered for lake level variation (around 2 m), the coast is made up of walls of buildings and structures directly facing the lake.

  This makes the computation of the water volume quite easy:

  $$\Delta Volume = Surface * \Delta Level \tag{7.1}$$

- As explained before in Sect. 3.3, inflows are not directly measured but they are computed backward moving starting from outflow and levels measurements. In the model inflows are taken as effective data that will be used for simulation.

- Flow rates taken are the mean effective flow rates over the day, they are considered constant for all the day analyzed.

- Dynamic lamination is neglected, still water level is taken horizontal as approximation, even if the lake is 46 km long.

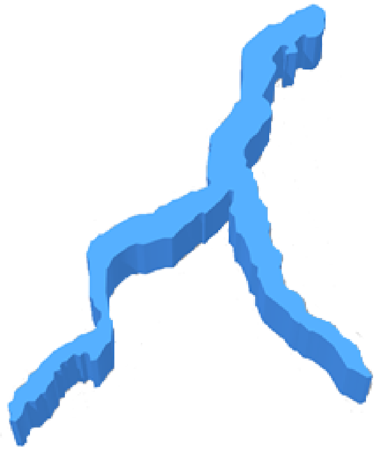


Figure 7.1: Lake Como control volume: approximation with vertical shores and horizontal surface

The continuity equation 7.2 describes the functioning of a linear reservoir, in this case Lake Como, with its process of filling and emptying [Becciu and Paoletti, 2010].

$$Q_{in}(t) - Q_{out}(t) = \frac{dW(t)}{dt} \tag{7.2}$$

Where:
- $W(t)$: volume stored at time t
- $Q_{in}(t)$: ingoing flow rate at time t
- $Q_{out}(t)$: outgoing flow rate at time t

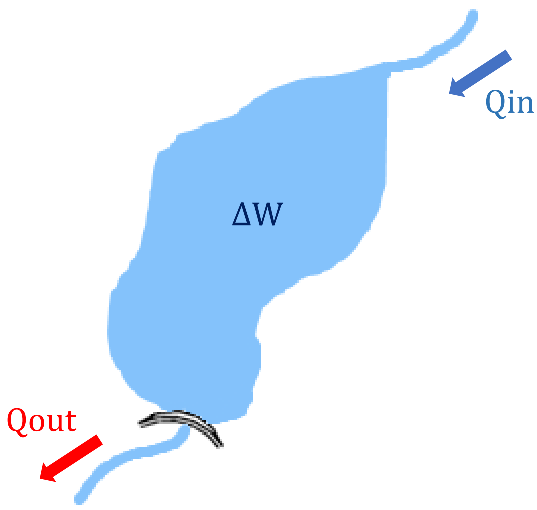


Figure 7.2: Schematic explanation of the continuity equation

Integrating the continuity equation over time it's computed the variation of water volume in the reservoir over time.

$$\int_0^t q_{in}(\tau) - \int_0^t q_{out}(\tau) = W_{in}(t) - W_{out}(t) + \Delta \tag{7.3}$$

$$W(t) = \Delta + W_{in}(t) - W_{out}(t) + W_s(t) \tag{7.4}$$

With:
- $W(t)$: volume stored at time t
- Δ: volume stored at time t_0 (initial instant)
- $W_{in}(t)$: cumulative ingoing volume till time t
- $W_{out}(t)$: cumulative outgoing volume till time t
- $W_s(t)$: cumulative spilled volume till time t

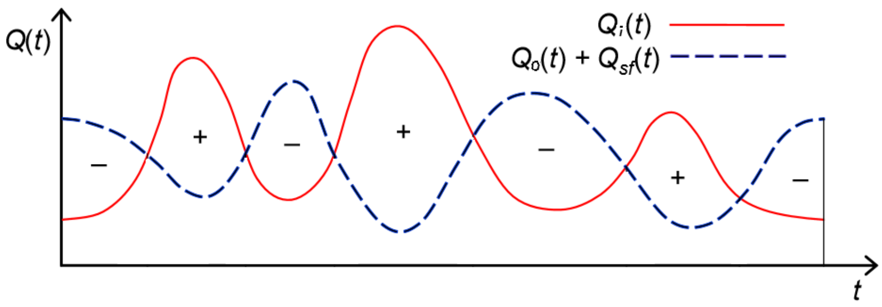


Figure 7.3: Schematic explanation of the continuity equation, ingoing and outgoing volumes

The chosen outflow from the regulation policy $Q_{out} m^3/s$ is kept constant for all the simulation period. As explained before, inflows are taken from the historical series, in the year prefixed starting from the day of forecast beginning.

When the simulator runs ingoing and outgoing volumes are computed daily and subtracted one from the other to compute levels trend:

$$\Delta W_{in}(i) = Q_{in}(i) * \Delta t \tag{7.5}$$

$$W_{in,cum}(i) = W_{in,cum}(i-1) + W_{in}(i) \tag{7.6}$$

Where:
- i: is i-th day
- Δt: is the number of seconds in one day $86400s$
- $\Delta W_{in}(i)$: is the inflow volume in i-th day
- $W_{incum}(i)$: is the cumulative inflow volume in i-th day

$$\Delta h_{in}(i) = \frac{W_{in}(i)}{S} \tag{7.7}$$

$$h_{in,cum}(i) = h_{in,cum}(i-1) + \Delta h_{in}(i) \tag{7.8}$$

With:
- $\Delta h_{in}(i)$: difference of incoming volume in i-th day
- S: lake surface $145 km^2$

The same equations are used for outflow, at the end of i-th day the lake level will be:

$$h(i) = h_{in,cum}(i) - h_{out,cum}(i) \tag{7.9}$$

Concession boundaries for regulation were described in Section 3.2.1, to stay on safe side is taken a margin of 40cm on both sides making a sort of "optimal band" for lake levels.

Table 7.1: Concession boundaries and "optimal band" for lake levels

Upper-bound concession	+120 cm
Lower-bound concession	-40 cm
Upper-bound "optimal band"	+80 cm
Lower-bound "optimal band"	0 cm

Once known the outflow and inflows, from historical series for the simulator and if the model is applied in reality from real-time data, the program computes lake level trend day-by-day using continuity equation.

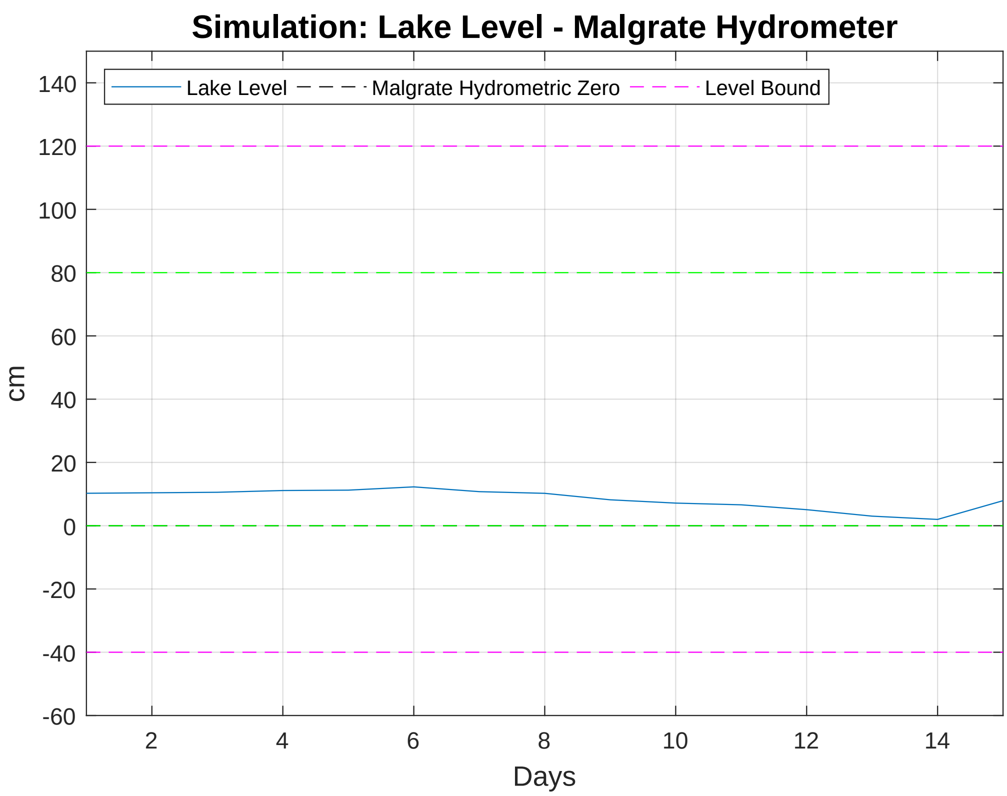


Figure 7.4: Lake level - Malgrate hydrometer, concession boundaries and "optimal band"

Two "warning signs" are inserted in the simulation, their purpose is of warning the decision maker when the lake level goes outside concession boundaries or the "optimal band".

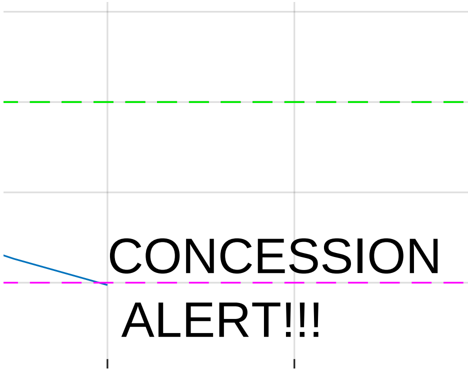


Figure 7.5: Concession alert

Figure 7.6: "Optimal band" alert

When lake level goes outside concession boundaries, the program stops and warns the decision maker to restart the program and to choose another Q_{out} basing on inflows forecasts starting from the day when concession alert lights up. If the lake level goes out the concession limit means that the policy chosen is wrong and not suitable to inflows.

When lake level goes outside the "optimal band", the program doesn't stop, because it is still inside concession boundaries, but warns the decision maker that the level begins to go outside the limits and suggests him to correct the Q_{out}, maybe redoing inflows future projections from that day.

The lamination volume provided by the lake is small, sudden flood events are difficult to be managed, for this reason is important to act in advance. The regulation tool designed in this work doesn't contain any s to weather forecasts, necessary to correctly prevent serious damages from floods. The tool deals with optimal regulation policy and management during a standard period, when critical events occur, the "warning signs" will inform the decision maker that something unusual is going on, Consorzio dell'Adda already has other tools for flood management, furnished by Progea Srl, by which can decide how to deal with the critical event.

For dry periods in summer the "Optimal band" alert is very useful when lake level is going down too fast. Since precipitations for a long time can be rare and modest, and considering that summer season is the part of the year in which water supply is fundamental for irrigation, it's very important to save water to be able to guarantee a minimum amount of water to all the users till the end of irrigation period. When the warning sign lights up, the decision maker has to re-evaluate the regulation policy for the lake, maybe decreasing Q_{out} to save water, as explained before the tool doesn't give a precise value of delivered outflow to be kept, gives a guideline for future inflows and relies on the decision maker's experience for the correction of the delivered water.

Below is reported an example of different regulation policies applied, with different Q_{out} related, all the simulations start on 09/04/2019 and is always taken a simulation time of 15 days (two weeks). It can be observed how differently behaves the program, giving "warning signs" when needed.

Example 1: $Q_{out} = 180 m^3/s$, starting day of the simulation 09/04/201

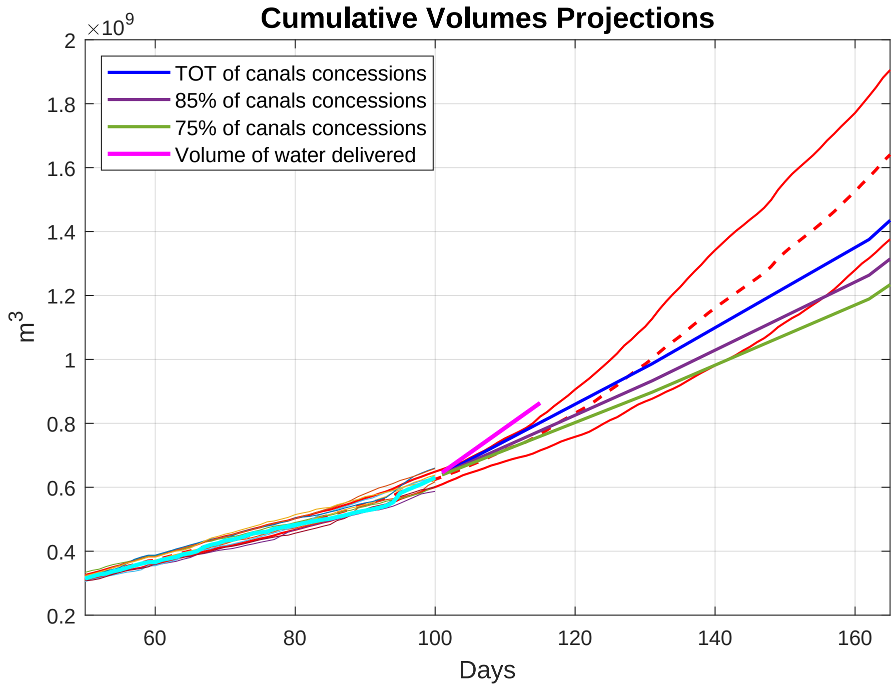


Figure 7.7: Example 1: Cumulative volumes projections

It can be noticed that delivered outflow is outside "red band" of inflows future projections and above cumulative volumes of canals concessions, this means that irrigation will be fully satisfied, moreover there will be more water than necessary, on the other side probably inflows won't be enough to sustain this amount of water delivered.

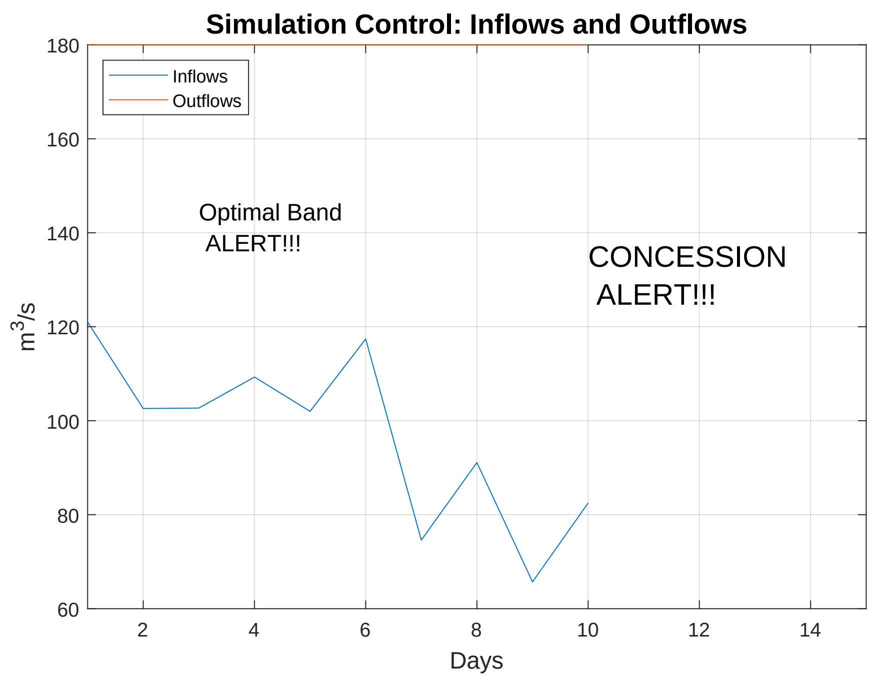


Figure 7.8: Example 1: Simulation control, inflows and outflows

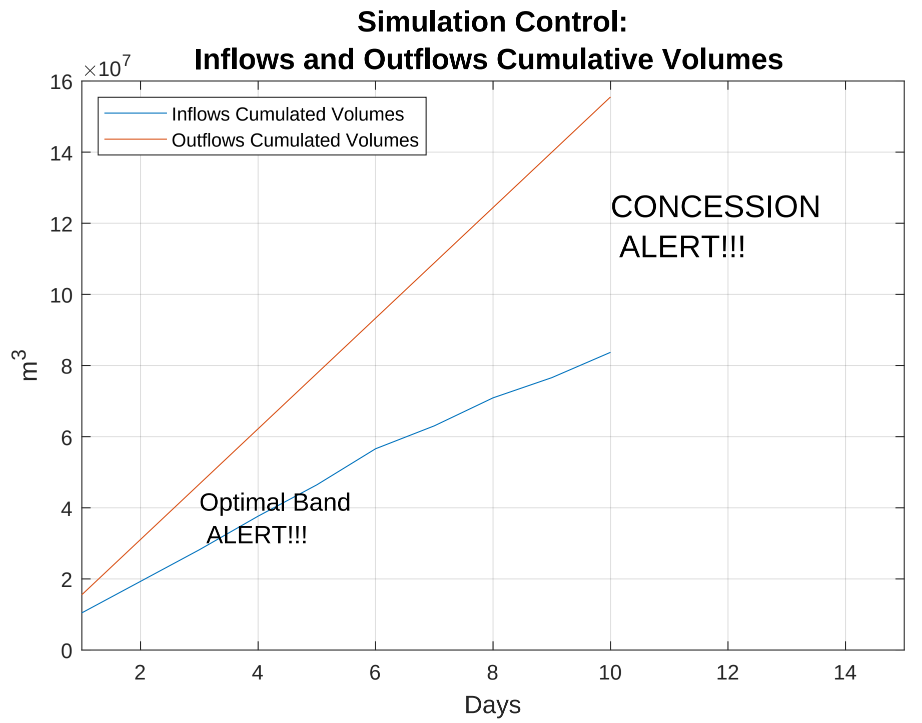


Figure 7.9: Example 1: Simulation control, inflows and outflows cumulative volumes

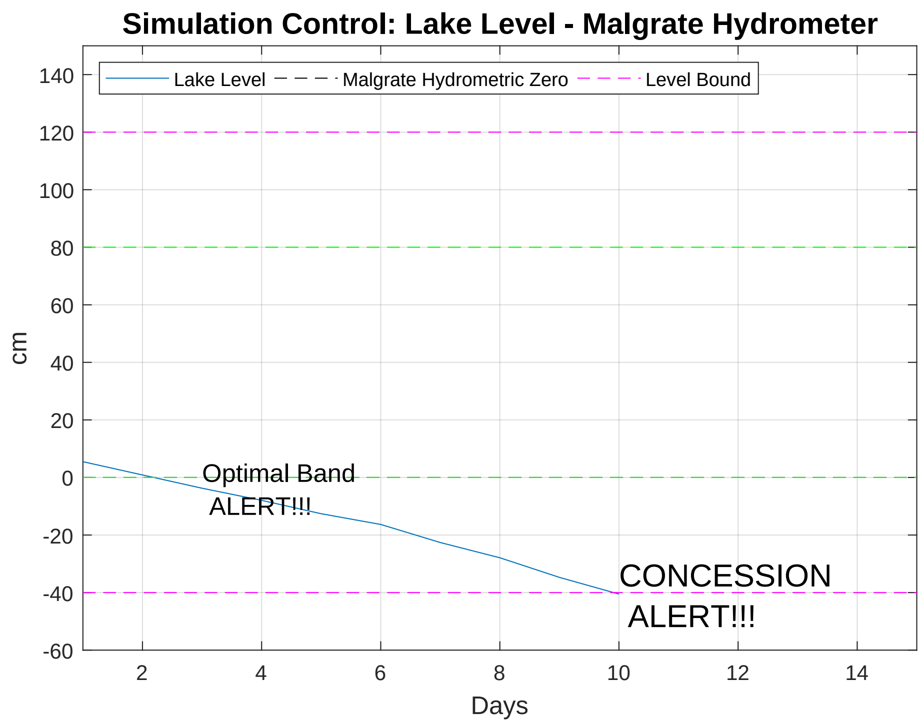


Figure 7.10: Example 1: Simulation control, lake level - Malgrate hydrometer

As it was predicted, inflows and outflows cumulative volumes continue to diverge, this means that lake level is rapidly decreasing, till the end of the simulation when concession alert sign lights on.

Example 2: $Q_{out} = 135m^3/s$, starting day of the simulation 09/04/201

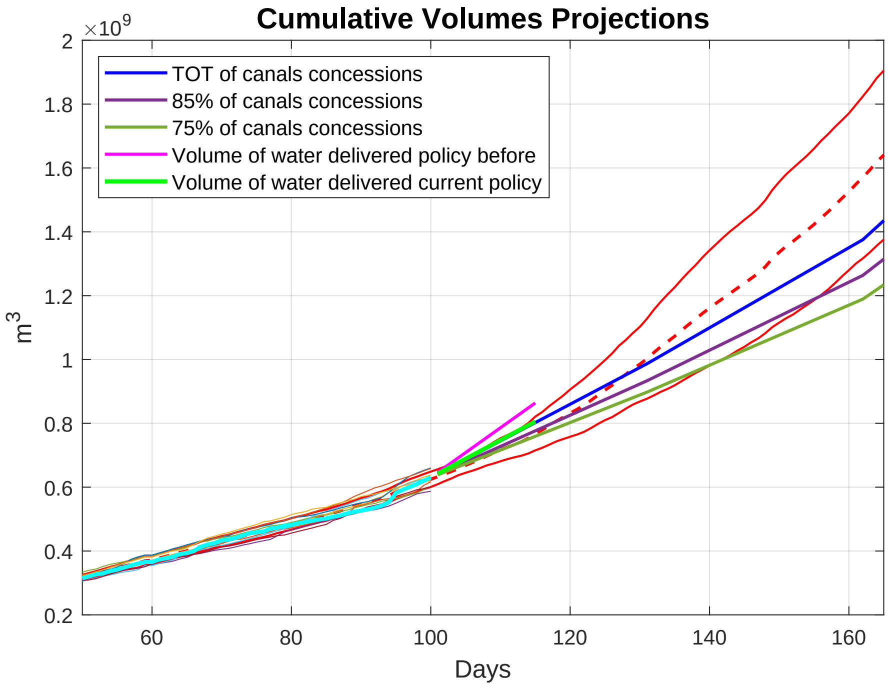


Figure 7.11: Example 2: Cumulative volumes projections

Differently from the previous policy, cumulative volumes are inside the "red band", actually on the edge, there is the possibility that inflows can sustain delivered outflow, but this is a long shot. Delivered outflow of $135m^3/s$ fully satisfies irrigation needs, totally of $123,2m^3/s$ in this period, and also leaves a margin about $10m^3/s$ for MIF in the case that tributaries are dry. It's a quite high outflow, but on the other hand Q_{out} can be corrected later whenever lake level is getting too low.

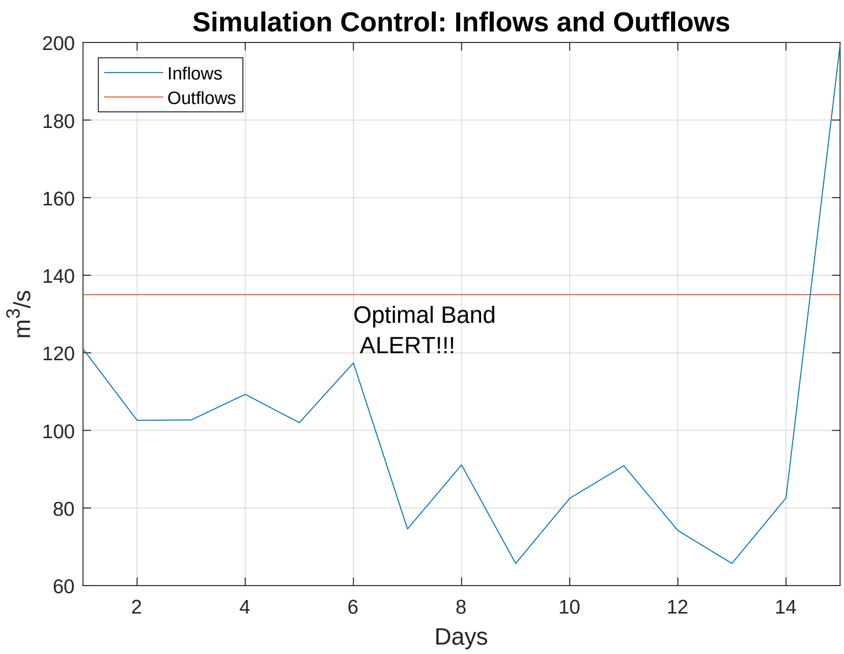


Figure 7.12: Example 2: Simulation control, inflows and outflows

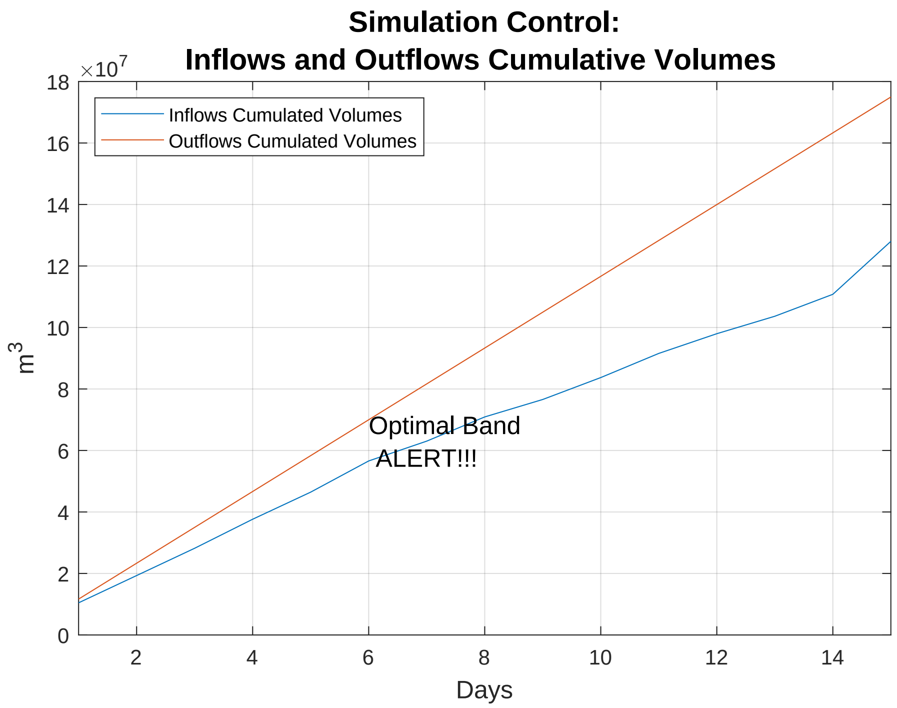


Figure 7.13: Example 2: Simulation control, inflows and outflows cumulative volumes

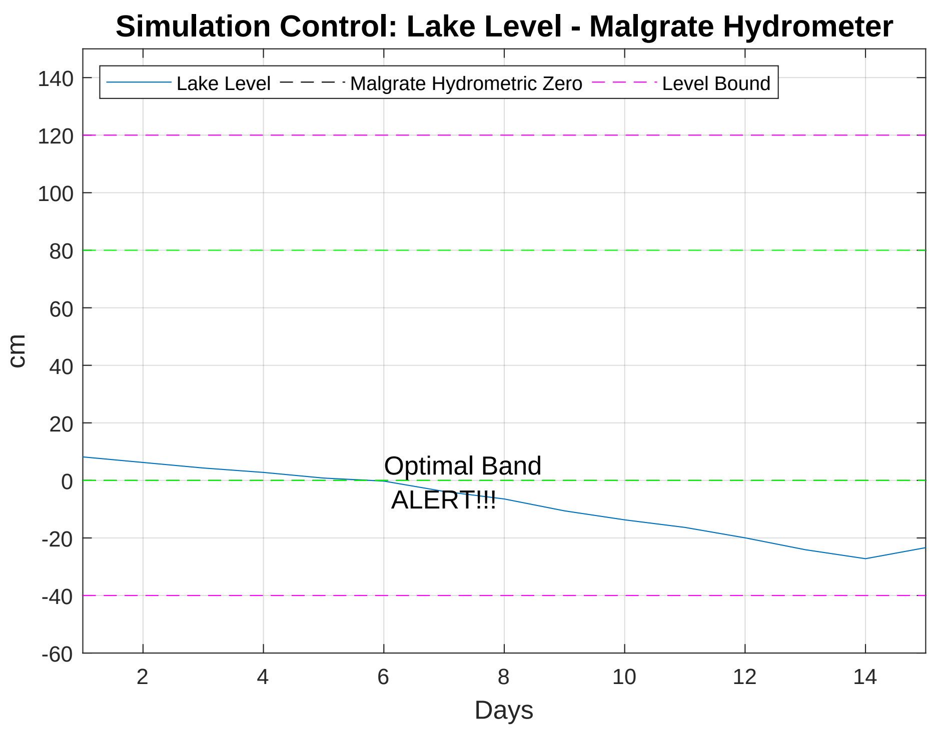


Figure 7.14: Example 2: Simulation control, lake level - Malgrate hydrometer

The "warning sign" alerts that the level starts to be too low, the management authority can choose to correct or not the regulation policy, thanks also to weather forecasts.

Example 3: $Q_{out} = 100m^3/s$, starting day of the simulation 09/04/201

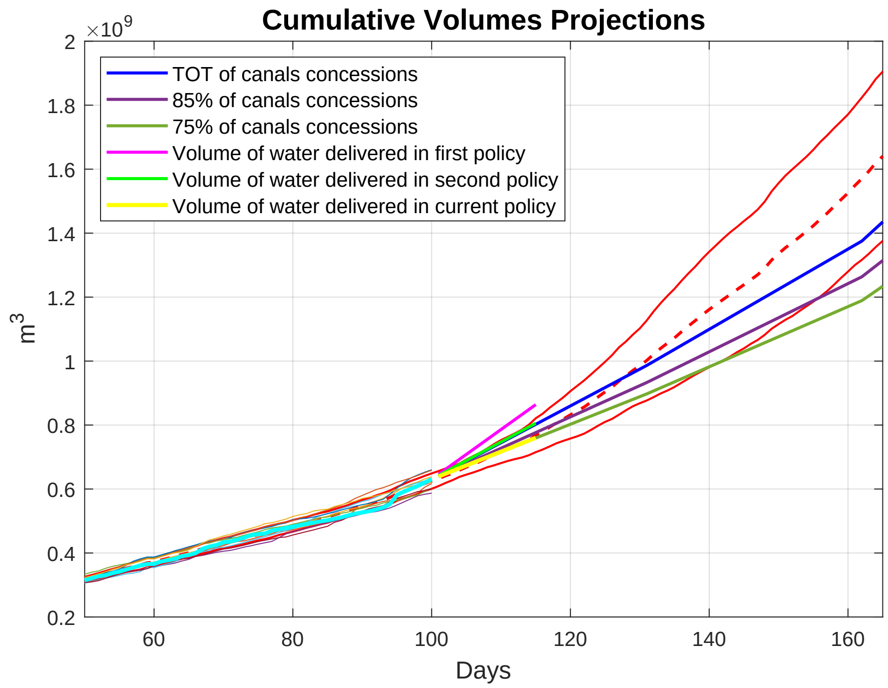


Figure 7.15: Example 3: Cumulative volumes projections

The last policy shown, is the most saving one, it corresponds to the 75% of canals concessions, a reasonable amount of water, according to NB curves in Section 5.3.2. It is reasonable to think that the policy will be ok for all the period, of 15 days, since the line of cumulative volumes for delivered outflow is in the middle of the "red band".

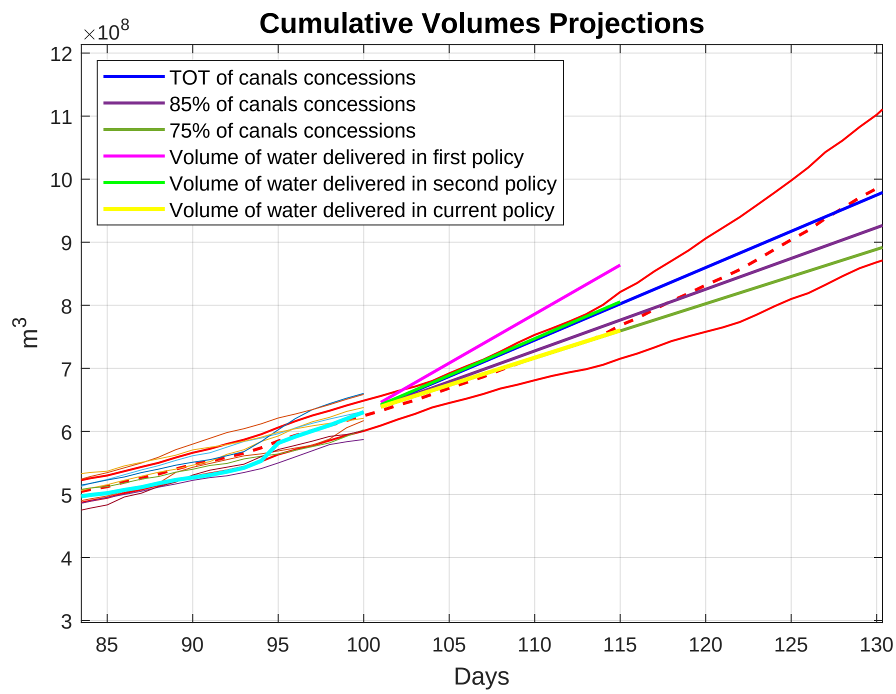


Figure 7.16: Example 3: Cumulative volumes projections, Zoom

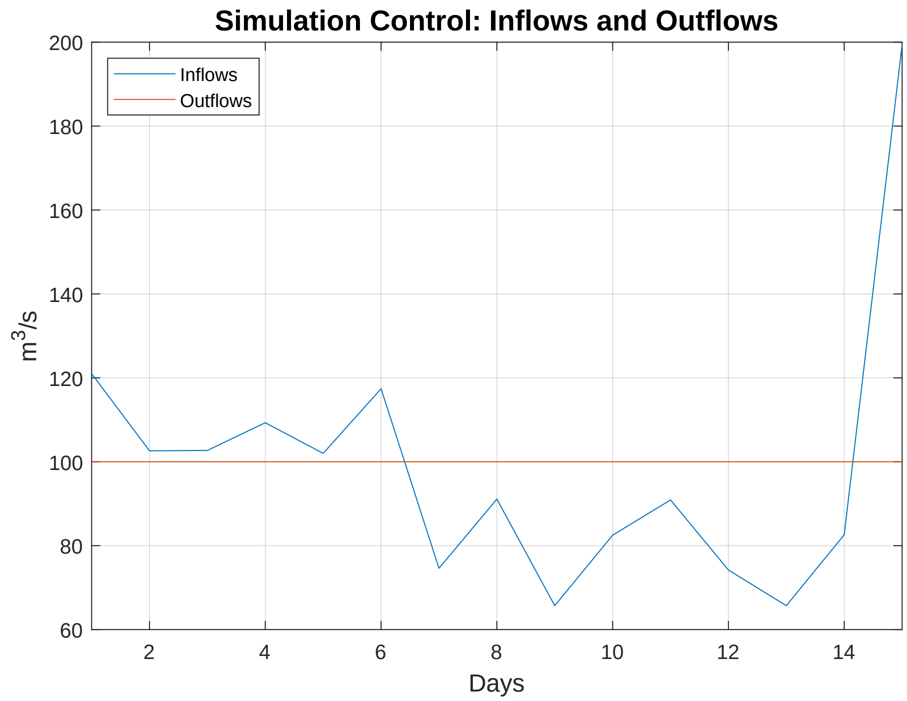


Figure 7.17: Example 3: Simulation control, inflows and outflows

The inflows and outflows cumulative volumes curve coincide at the end of the projection period, this means that lake level will be almost the same between the beginning and the end.

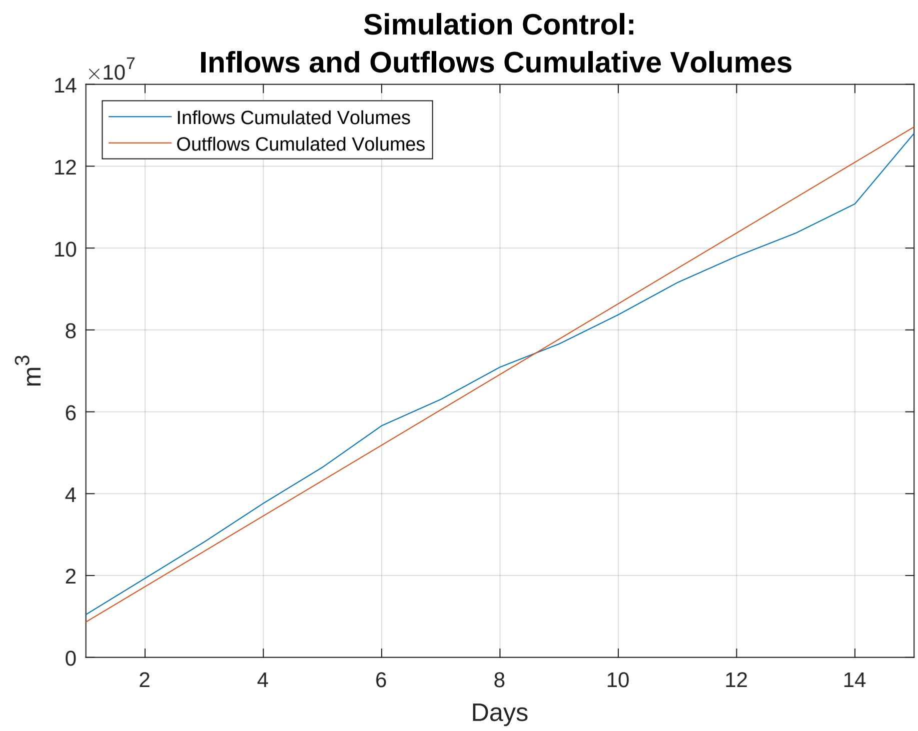


Figure 7.18: Example 3: Simulation control, inflows and outflows cumulative volumes

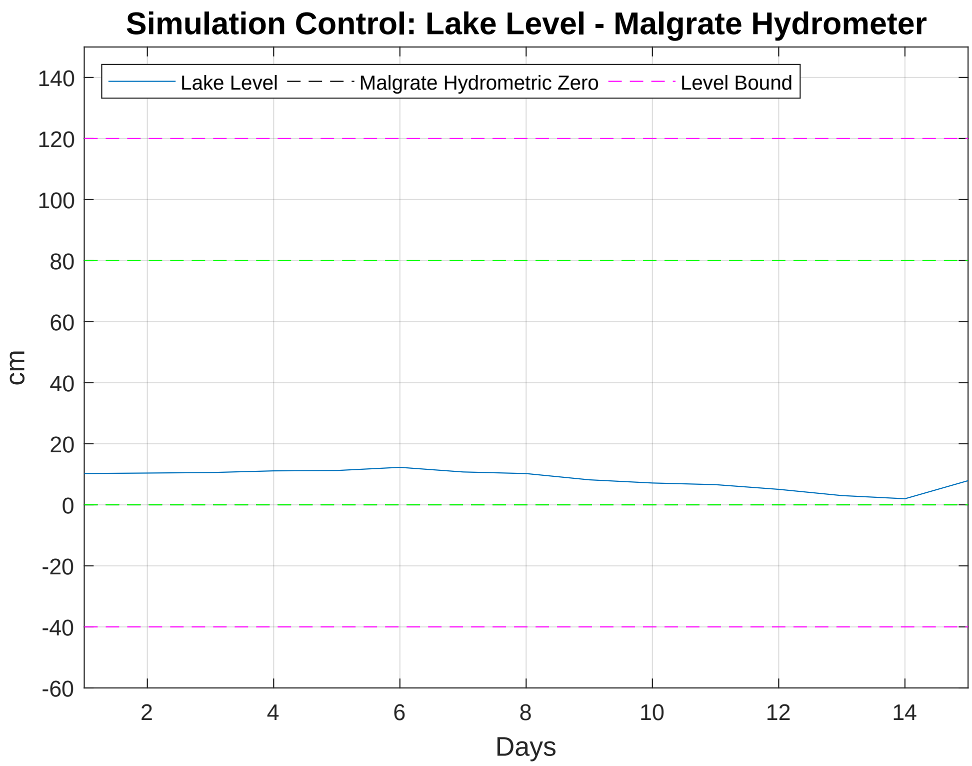


Figure 7.19: Example 3: Simulation control, lake level - Malgrate hydrometer

7.2 Application for downstream river Adda water management

The optimal regulation policy chosen is finally tested for river Adda water management. For the simulation of downstream river basin it's used river Adda approximated simulator, introduced in Section 4.3, while allocation policy applied is the same of Consorzio dell'Adda, following the principle of proportionality, as discussed in Chapter 5.

The delivered outflows compared are the same of the previous section:

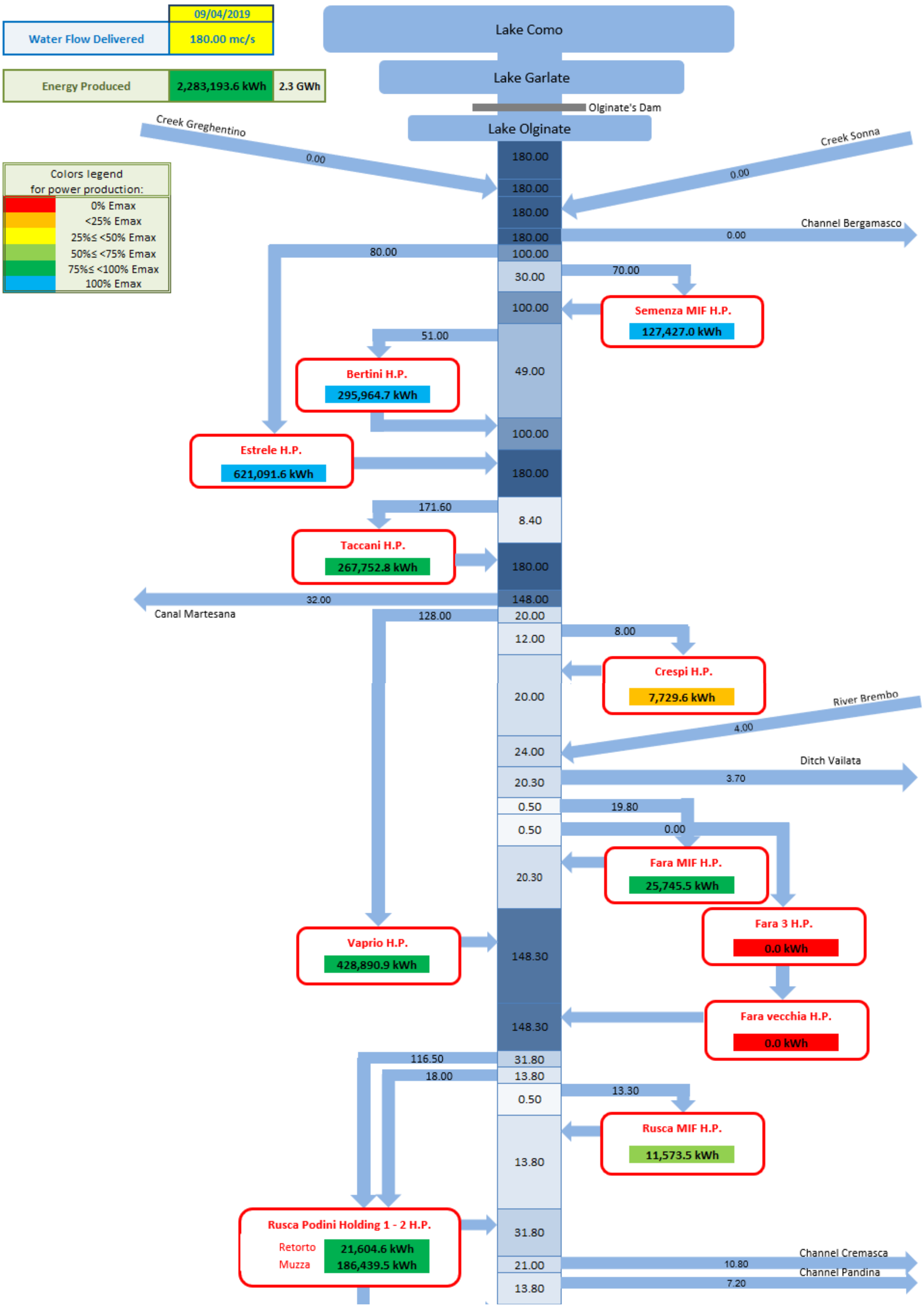
1. $Q_{out} = 180m^3/s$: water in excess, canals concessions fully satisfied, but this outflow can't be kept for a long period.

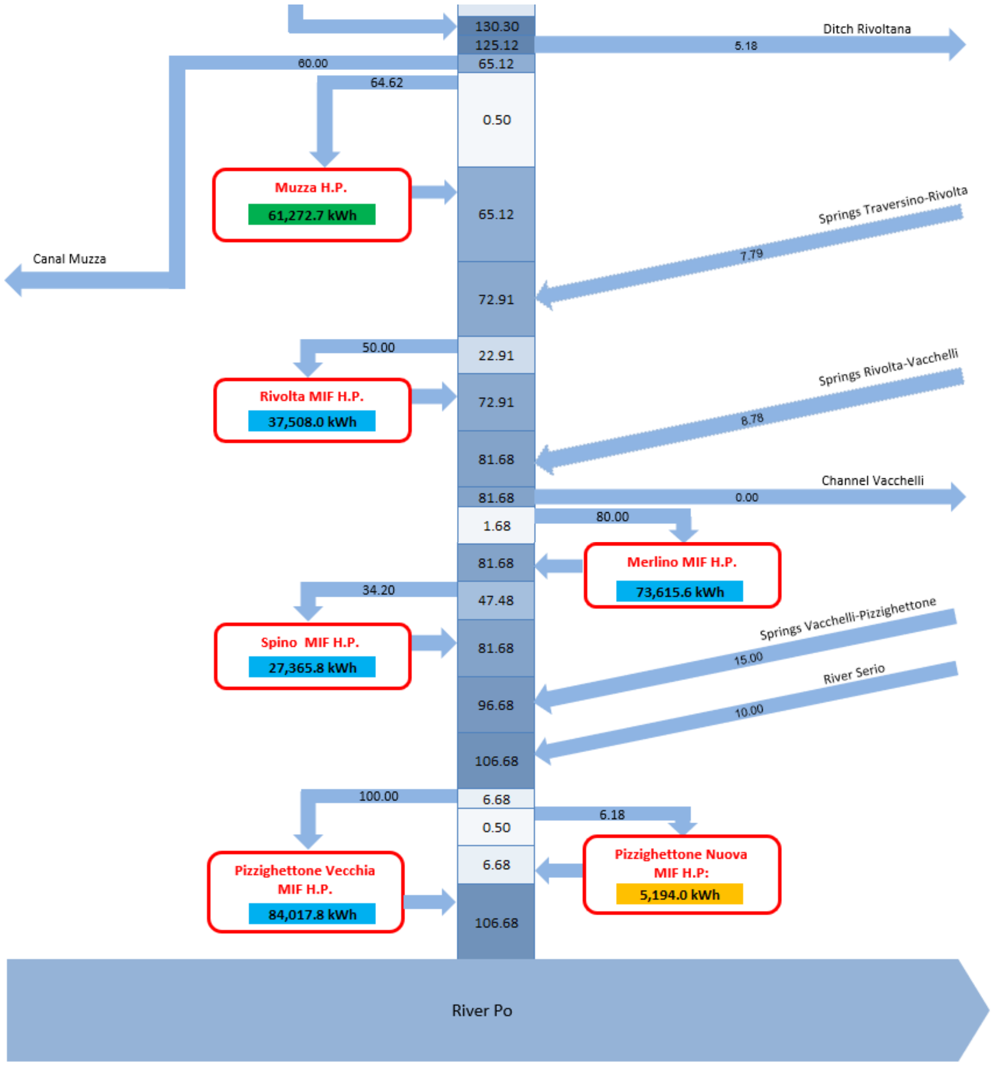
2. $Q_{out} = 135m^3/s$: canals concessions fully satisfied, it's risky to keep this outflow all the time, but on the other hand the policy can be changed whenever the levels goes too low.

3. $Q_{out} = 100m^3/s$: canals concessions satisfied for the 75% of the concession. Is the most saving policy that almost surely can be kept for all the period.

Remembering that the simulation it's done starting from 09/04, so the sum of all the canals concession is $123.2m^3/s$.
In summer period, the situation will be more critical, since irrigation needs will be greater and river runoff lower, the power production will be less; for these reasons it's important to save water during wet periods, keeping a quite high lake level, so that we can ensure a decent amount of water in the river for all dry period.

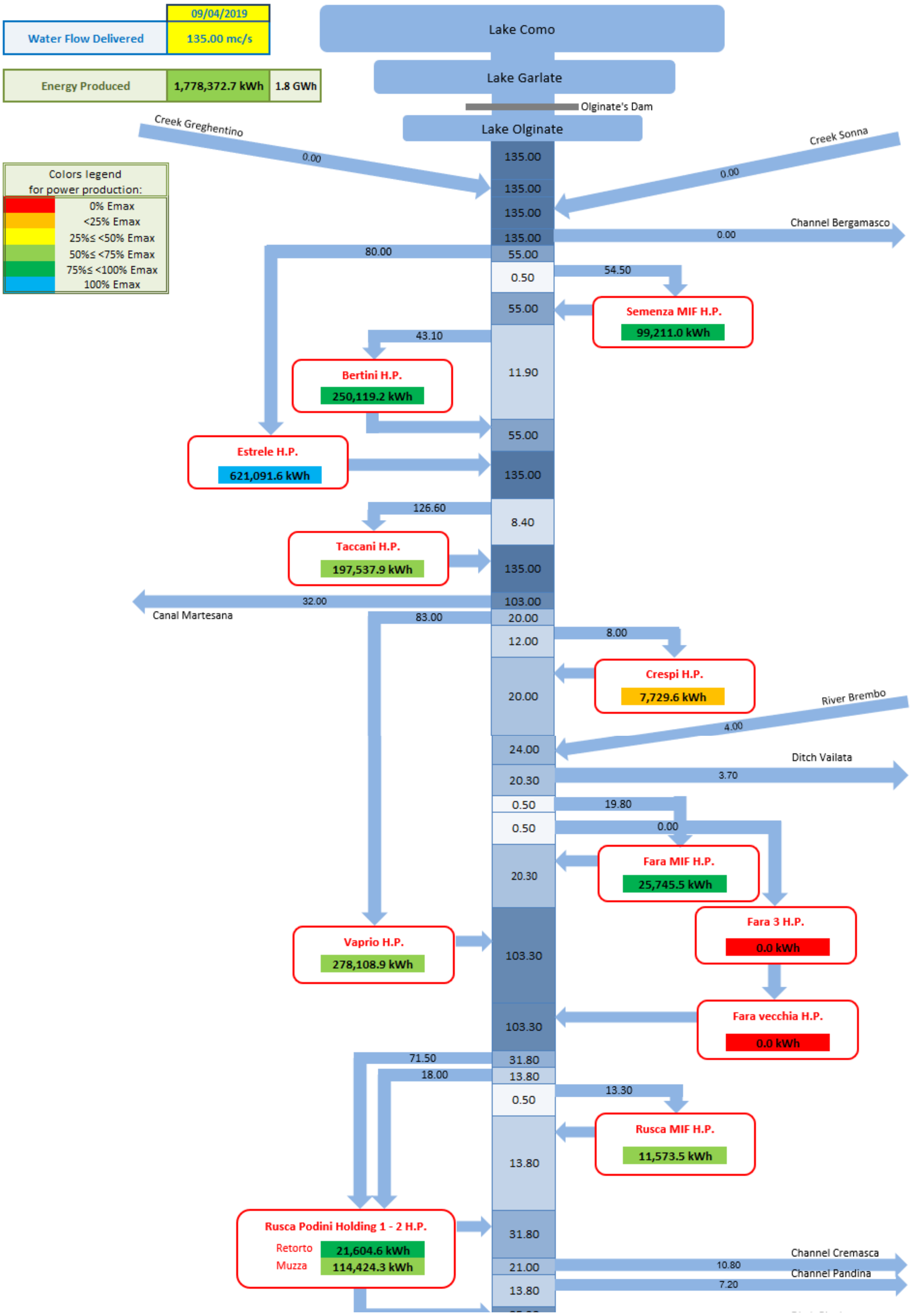
Figure 7.20: Example 1: $Q_{out} = 180 m^3/s$, Adda river system simulator

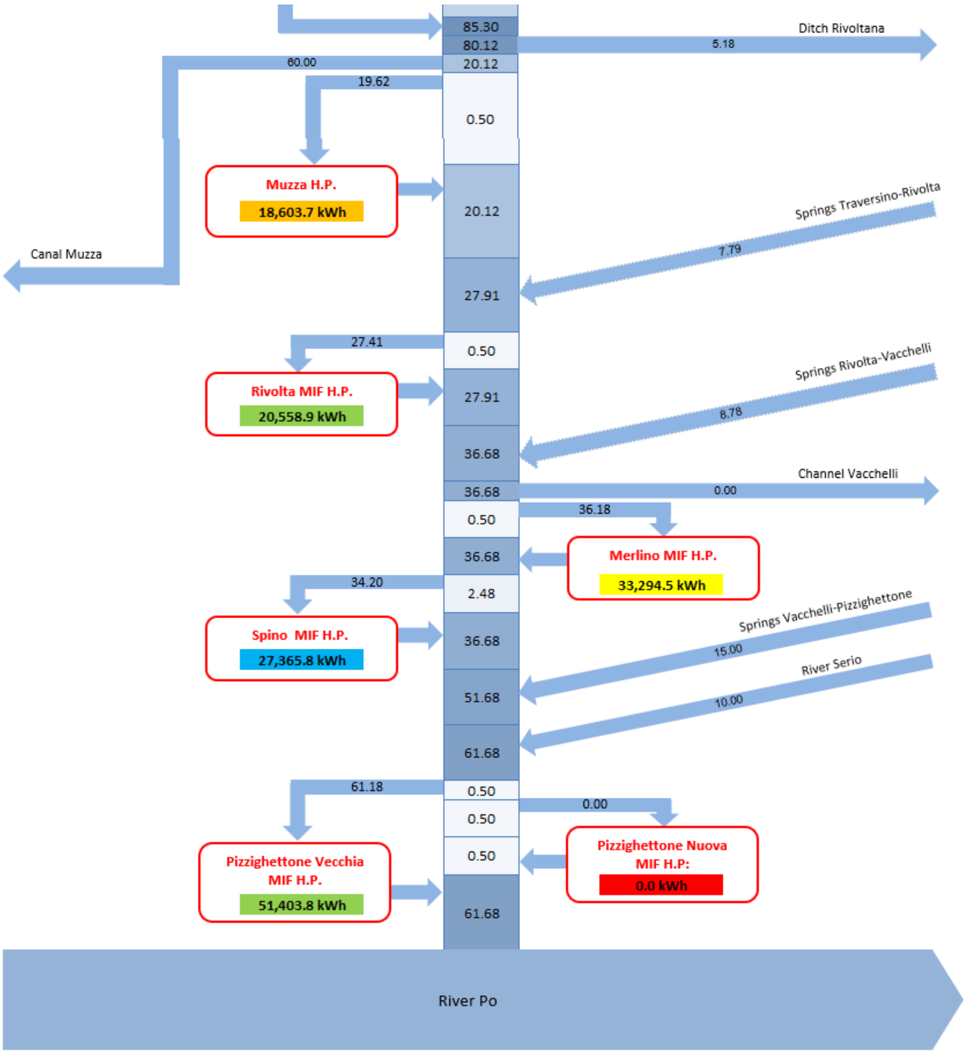




Summary of the power production in course:					
	Q [m^3/s]	ΔH [m]	P[kW]	E [kWh/day]	% max
Esterle H.P.	80.0	38.8	25,878.8	621,091.6	100%
Semenza MIF H.P.	70.0	9.1	5,309.5	127,427.0	100%
Bertini H.P.	51.0	29.0	12,331.9	295,964.7	100%
Taccani H.P.	171.6	7.8	11,156.4	267,752.8	95%
Vaprio H.P.	128.0	16.8	17,870.5	428,890.9	95%
Crespi H.P.	8.0	4.8	322.1	7,729.6	13%
Fara MIF H.P.	19.8	6.5	1,072.7	25,745.5	79%
Fara Vecchia H.P.	0.0	4.9	0.0	0.0	0%
Fara 3 H.P.	0.0	1.9	0.0	0.0	0%
Rusca MIF H.P.	13.3	4.4	482.2	11,573.5	65%
Rusca-Retorto H.P.	18.0	6.0	900.2	21,604.6	86%
Rusca-Muzza H.P.	116.5	8.0	7,768.3	186,439.5	86%
Muzza MIF H.P.	64.6	4.7	2,553.0	61,272.7	81%
Rivolta MIF H.P.	50.0	3.8	1,562.8	37,508.0	100%
Merlino MIF H.P.	80.0	4.6	3,067.3	73,615.6	100%
Spino MIF H.P.	34.2	4.0	1,140.2	27,365.8	100%
Pizzighettone old MIF H.P.	100.0	4.2	3,500.7	84,017.8	100%
Pizzighettone new H.P.	6.2	4.2	216.4	5,194.0	5%
TOT	**1011.2**	**163.4**	**95,133.1**	**2,283,193.6**	**88%**

Figure 7.21: Example 2: $Q_{out} = 135m^3/s$, Adda river system simulator

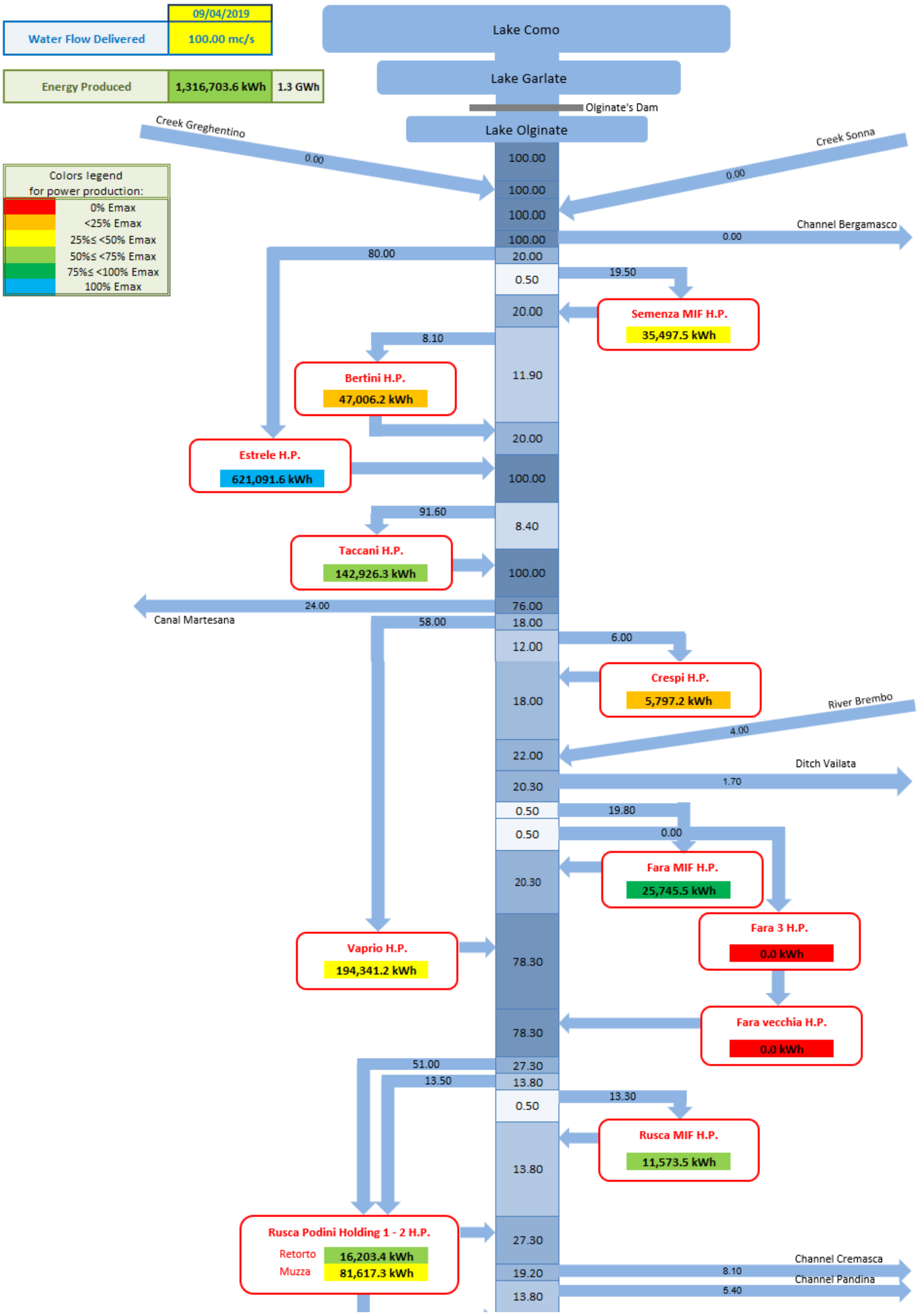


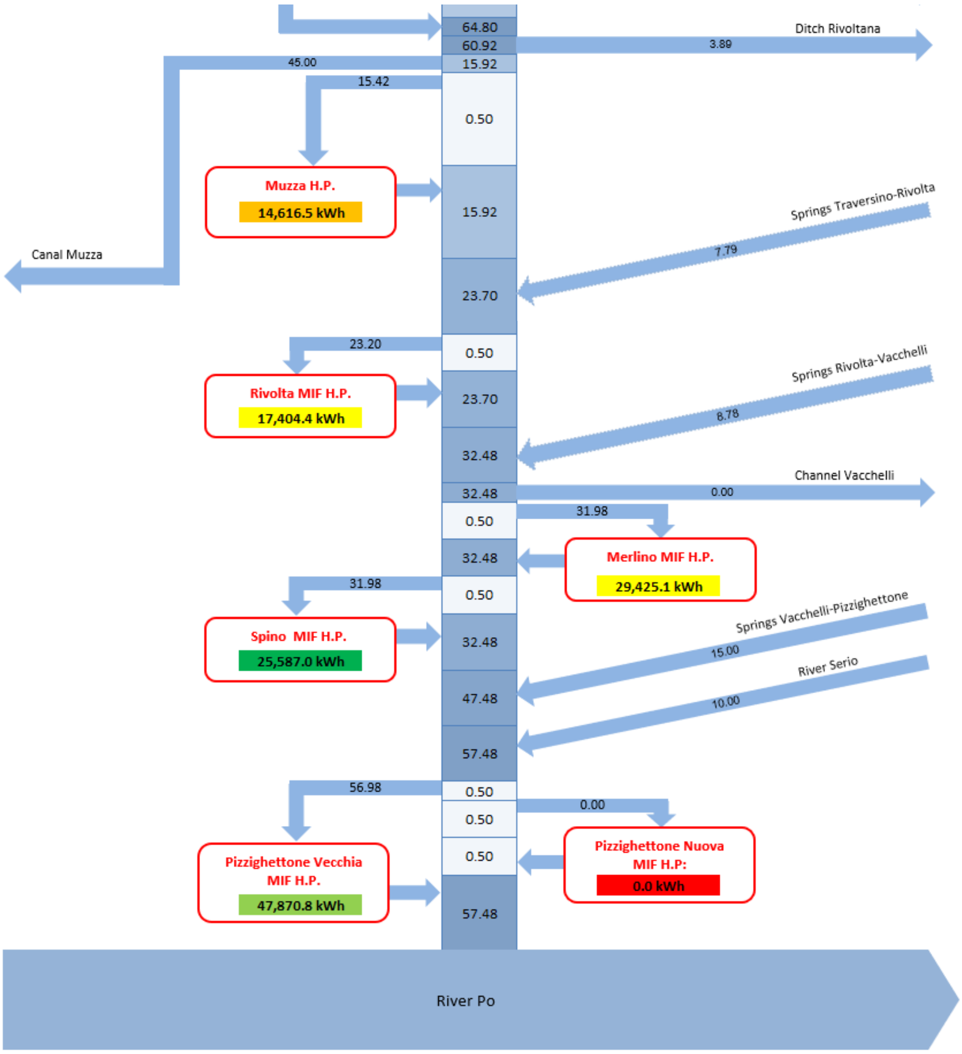


Summary of the power production in course:					
	Q [m^3/s]	**ΔH [m]**	**P[kW]**	**E [kWh/day]**	**% max**
Esterle H.P.	80.0	38.8	25,878.8	621,091.6	100%
Semenza MIF H.P.	54.5	9.1	4,133.8	99,211.0	78%
Bertini H.P.	43.1	29.0	10,421.6	250,119.2	85%
Taccani H.P.	126.6	7.8	8,230.7	197,537.9	70%
Vaprio H.P.	83.0	16.8	11,587.9	278,108.9	62%
Crespi H.P.	8.0	4.8	322.1	7,729.6	13%
Fara MIF H.P.	19.8	6.5	1,072.7	25,745.5	79%
Fara Vecchia H.P.	0.0	4.9	0.0	0.0	0%
Fara 3 H.P.	0.0	1.9	0.0	0.0	0%
Rusca MIF H.P.	13.3	4.4	482.2	11,573.5	65%
Rusca-Retorto H.P.	18.0	6.0	900.2	21,604.6	86%
Rusca-Muzza H.P.	71.5	8.0	4,767.7	114,424.3	53%
Muzza MIF H.P.	19.6	4.7	775.2	18,603.7	25%
Rivolta MIF H.P.	27.4	3.8	856.6	20,558.9	55%
Merlino MIF H.P.	36.2	4.6	1,387.3	33,294.5	45%
Spino MIF H.P.	34.2	4.0	1,140.2	27,365.8	100%
Pizzighettone old MIF H.P.	61.2	4.2	2,141.8	51,403.8	61%
Pizzighettone new H.P.	0.0	4.2	0.0	0.0	0%

TOT	**696.4**	**163.4**	**74,098.9**	**1,778,372.7**	**68%**

Figure 7.22: Example 3: $Q_{out} = 100m^3/s$, Adda river system simulator





Summary of the power production in course:					
	Q [m³/s]	ΔH [m]	P[kW]	E [kWh/day]	% max
Esterle H.P.	80.0	38.8	25,878.8	621,091.6	100%
Semenza MIF H.P.	19.5	9.1	1,479.1	35,497.5	28%
Bertini H.P.	8.1	29.0	1,958.6	47,006.2	16%
Taccani H.P.	91.6	7.8	5,955.3	142,926.3	51%
Vaprio H.P.	58.0	16.8	8,097.5	194,341.2	43%
Crespi H.P.	6.0	4.8	241.6	5,797.2	10%
Fara MIF H.P.	19.8	6.5	1,072.7	25,745.5	79%
Fara Vecchia H.P.	0.0	4.9	0.0	0.0	0%
Fara 3 H.P.	0.0	1.9	0.0	0.0	0%
Rusca MIF H.P.	13.3	4.4	482.2	11,573.5	65%
Rusca-Retorto H.P.	13.5	6.0	675.1	16,203.4	64%
Rusca-Muzza H.P.	51.0	8.0	3,400.7	81,617.3	38%
Muzza MIF H.P.	15.4	4.7	609.0	14,616.5	19%
Rivolta MIF H.P.	23.2	3.8	725.2	17,404.4	46%
Merlino MIF H.P.	32.0	4.6	1,226.0	29,425.1	40%
Spino MIF H.P.	32.0	4.0	1,066.1	25,587.0	94%
Pizzighettone old MIF H.P.	57.0	4.2	1,994.6	47,870.8	57%
Pizzighettone new H.P.	0.0	4.2	0.0	0.0	0%
TOT	**520.3**	**163.4**	**54,862.6**	**1,316,703.6**	**51%**

The simulator reveals that Fara 3, Fara nuova and Pizzighettone nuova hydropower plants work only for delivered outflow bigger than $180m^3/s$, this isn't properly true. Adda Energy Srl owes all the three plants in Fara Gera d'Adda (BG) and Edison Spa owes both Pizzighettone (CR) hydropower plants, the internal strategy of a company energy for itself power production is unknown; the tool gives an idea of the energy produced in total over Adda river system, but doesn't specify those cases.

As can be seen in the various policies, a higher outflow is more difficult to be kept for a long period, water runs out quickly in dry periods and in an heartbeat the lake can't supply the irrigation and hydropower system. Especially in summer period, a wise way for managing water is to keep a quite low outflow, even if there is water availability, in order to save it and keep it constant for more time. The hydropower plants have a limited installed capacity, when river runoff is higher than the flow rate exploitable by the plant it is useless for what concerns power production; on the other hand turn on a power plant for a very slow flow rate is more a cost than a profit. For these reasons keeping a constant medium river runoff for a quite long period is more efficient than another policy, in this way the runoff is always exploited for power production.

CHAPTER 8

CONCLUSIONS AND FUTURE DEVELOPMENTS

The program conceived gives an overview on the whole water catchment, starting from the upstream Lake Como and it's regulation and continuing with water management downstream over Adda river system.

The model is quite approximated and tries to circumscribe the area of interest. In order to have a more complete model it would be necessary to take into account also Valtellina Valley area, with its alpine reservoir and their system of regulation, the groundwater body staying below the surface of the whole region, the auxiliary system of channels and intakes, sometimes not properly known and other variable factors like municipal and industrial users, changing legislation, environmental incentives...

These make the system very complex and several variables more, the choice is to reduce river system to the essential, since the objective is to give a tool useful for a general planning of a regulation policy and for a checking over the system easy to understand.

To be even more precise, eventual gaps in mass balance along river path could be corrected by simulation. Taking data series of river runoff in different sections and historical data from all the users, both hydroelectric and agricultural; it would be a further development to compare day-by-day simulated power production and effective one, simulated river runoff and registered one, simulated withdrawals by irrigation channels and effective ones and so on... This would solve hydraulic balance problems in the simulator, many of them due to water exchange between surface and groundwater system, moreover it would make more precise the computation for hydropower production.

The hope is to have the chance to apply optimal policy, planned with the regulation tool, to the real case, thanks to Consorzio dell'Adda support and after improving the interface for an easier understanding.

I would like to remark that the program developed does not inspire to be an optimization model, but instead aims to be a useful tool in the hands of the decision maker for the planning of an optimal regulation policy for Lake Como and water management of Adda river basin.

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Dimensioni Profondità

Viabilità Strade Sentieri

Navigabilità Prima

Commercio Setifici Manufatti ferro Cantieri
Como cave Sassi

Trasporti Barconi Sassi

Turismo Ville Storiche

Invasioni Spagnoli

Pesca + Pesce Persico Lavarello Agone
Luccio Trota Alborelle Cavedano tinca carpe
Luccio Perca Anguilla Scardola Rugo ecct
Bottatrice

Venti Predominanti Mattino Tivano Pomeriggio Breva
Primo Bacino

Zone Campeggi Domaso Gravedona Alto Lago

Carlo