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DOCTORAL THESIS

Optimisation of both energy use and pumping costs in water distribution networks with several water sources using the setpoint curve

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Abstract

Pump system optimisation implies both investment cost optimisation and operational costs. These are closely related and depends fundamentally on three aspects: a) optimum sizing, b) optimum selection, and c) optimum operation. However, a suitable characterisation of the water distribution networks is fundamental previous to addressing these three aspects. This characterisation is usually done through obtaining of the system head curves (SHCs).

Often, SHCs are associated with the resistances curves (RCs). The RCs refer to the flow and pressure head needed in each pumping station to fulfil the user’s demand. For that purpose, the resistance of the system (i.e. head losses, static lift, etc.) derived from the spatial and temporal variation of the network demand as well as the location of the discharge points must be overcome. As the demand is subject to human dynamics, it is highly variable and therefore such variability results in multiple resistant curves. Demand variation is transmitted to the network through the position of the resistance elements of consumers (i.e. valves, faucets, etc). In that sense, the resistance generated by the user adjusts to the pressure and flow necessities in each point of consumption. The difficulty in considering the variation of the system’s resistance generated by demand or, in other words, by the consumer makes hard to calculate all the points of the RCs

Theoretically, pumping systems are designed, selected and optimised depending on the operating points obtained by the intersection between three curves: a) resistance curve obtained for maximum demand, b) resistance curve obtained for minimum demand and c) pump performance curve. However, what is done is proposing a pumping system and obtaining its operating points according to the demand and pressure conditions of the network. Sometimes, the pumping system is selected from a set of alternatives, i.e. a search of the system that fits better the work conditions of the network is done. This process has been widely studied through the different mathematical optimisation models: classic (linear, non-linear, dynamic, quadratic, stochastic, etc.) and metaheuristic (evolutive algorithms, simulated annealing, ant colony, etc.). Nevertheless, the problem is that the pumping systems are designed and selected based on the network’s most critical operating point (i.e. maximum demand and minimum pressure) but are optimised leaving aside the fact that when the water consumption is lower so is the required energy. Thus, neither it is possible to quantify the energy excess that involves pumping operation for a lower demand than the maximum, regarding the minimum energy required. In that context, operating costs may be increased unnecessarily. This last aspect can be decisive when choosing one pumping systems or another different. Therefore, though traditional methods optimise the operation of the pumping systems, this process is incomplete since the real requirements of networks are not properly defined. In that way, the solution to

this problem will enable not only a better estimation and optimisation of the pumping operating costs but also for a better sizing and selection of pumping systems.

The problem can be approached differently by using the setpoint curve (SC). The SC is the second type of SHCs, poorly studied so far, and from which all its points can be calculated relatively easily. It refers to the flow and pressure head required in a supply node (for study purposes, a pumping station) to set a certain pressure in a reference node of the network. The reference node usually is the one with the lowest pressure, known as the critical node. The set pressure is the minimum necessary stablished by corresponding regulations. Hence, by maintaining the minimum pressure at the critical node, pressure requirements in the rest of the network nodes are achieved. This new perspective constitutes a fundamental part of the present thesis. In that context, the aim of this work consists in the formulation of a methodology for the optimisation of both energy use and pumping costs in water distribution networks with several sources of supply by using the SC.

Up to now, the process to calculate SCs had been studied only for networks with two pumping stations, by using a fixed flow distribution among supply sources, for non-pressure-driven demands and without considering available storage capacity. Therefore, one of the objectives of this work is to extend the SCs calculation methodology to overcome the limits mentioned. Usually, SCs are used to optimise, from the energetic perspective, the operation of the pumping systems by adjusting as much as possible the performance curve of the pumping system to the SCs calculated. However, when the flow distribution supplied by the pumping stations changes, so do the SCs and, the energy needs in each station. Thus, the present work also studies the way of obtaining the optimum flow distributions that minimises energy requirements for each network demand. These distributions allow converging towards the optimum SC of each pumping station. Besides, the optimisation method is developed for networks with multiple pumping stations, with pressure-driven demands (PDD) and non-pressure-driven demands (NPDD), and without storage capacity.

To obtain the optimum flow distribution two methods are proposed, one discrete (D-M) and other continuous (C-M). In the D-M, the flow distribution is assumed as a discrete variable, so defining a finite set of distributions previous to its application is needed. The distributions are percentage values and apply for each value of the network demand. Therefore, for each value of the demand and through an objective function (OF), all distributions are assessed. The outcomes are the energy values that each pumping station requires for each distribution and demand. Hence, it is possible to construct energy curves for each pumping station depending on the distribution and the demand. In this way, the minimum energy value indicates the optimum flow distribution for a certain demand. Furthermore, when the optimum flow distribution is reached, a point of the SC for each pumping station is also obtained. The point corresponds to the flow and pumping head given in the distribution. In this context, for a specific number of network demands, the same number of optimum flow distributions and one optimum SC per pumping station are obtained. In the case of the C-M the distribution is assumed as a continuous variable, so it is no longer necessary to obtain the energy curves for the different distributions by discretising the flow. This is because the optimum flow distribution is given by applying direct search optimisation algorithms. In this study, Hooke & Jeeves, and Nelder & Mead algorithms were implemented. The C-M is more precise than the D-M and allows addressing problems with a greater number of dimensions. On the contrary, in the D-M, the number of calculations increases exponentially when the number of pumping stations increases. Thus, the discretisation of the flow distribution turns much more complex.

Next step in the research is considering the implications of energy tariffs and other relevant costs, such as water production, on the optimal flow distribution. For this purpose, the C-M developed for the energy optimisation is used and costs of pumping and water production are included in the objective function. This enables reaching the optimum distribution attending the cost of the energy used by each pumping station. Thus, it is possible to also obtain the most economic SCs. Equal to energy optimisation, the said process is applicable to networks with multiple pumping stations, PPD and NPDD, but without storage capacity.

In the final part of this work, the inclusion of storage tanks in the process of optimisation and calculation of the least-cost SCs is addressed. This implies modifying the calculation methodology of SCs. To do so, two penalty costs are added to the same cost function when tanks are not considered. In that sense, the not compliance with pressure and storage volume are penalised. The optimisation is performed by means of the evolutive algorithms, Differential Evolution and the Hybrid Algorithm. It has to be noted that in the previous optimisation cases (i.e. energy and costs in networks without tanks), pressure penalisation was not included. The reason is that the calculation process of the SC implicitly guarantees to maintain the value of the minimum pressure on the critical node for each simulated scenario. Nevertheless, when tanks are considered, the pressure on the critical node cannot be the same over the whole simulation period. In fact, the pressure will vary depending on the tank levels and if tanks are filling or emptying. Thus, it is no possible to guarantee that the value of the minimum pressure maintains throughout the entire simulation. Hence, to compute the SCs maintaining an equal or greater pressure to the minimum permitted is pursued, always tending towards to the minimum possible. Regarding the storage penalisation cost, it is considered when at the end of the simulation the storage volume is less than the initial volume of the tanks. Moreover, penalisation costs are not fixed, but proportional to the default of the conditions, facilitating the search for an optimum global solution.

It has to be pointed out that capital costs when existing pumping stations need to be replaced have not been included. Therefore, this work is suitable only for the optimisation of pumping systems that are designed from scratch. In the case of existing systems, the optimal SCs will be useful as long as they can be set at pumping system with a minimum efficiency expected.

To demonstrate the developed methodologies, five distribution networks are studied under different functioning conditions: TF, Catinen, COPLACA, Anytown and Richmond. The first three (TF, Catinen and COPLACA) are networks without tanks that have been used for the study of energy and cost optimisation in networks without storage capacity. Some of the studied conditions are: PDD and NPDD, a variable number of available pumping stations, limitations on flow supply, etc. Besides, in TF network also pumps have been selected as a demonstrative example of the final application of the optimal SCs. Anytown and Richmond networks have been used for the study of energy and cost optimisation in networks with storage capacity. Some of the distinctive features studied are: variable number of tanks, booster pumping stations, a variable level at the suction of pumping stations, etc. In none of the networks have multiple operating conditions such as firefighting flow, seasonal demand curves, etc. been considered. The effect on the reliability due to removal of pumping systems or tanks has not been considered either. These aspects require further investigation and are complementary to the pumps sizing and selection.

Once the most economic flow distributions and SCs are obtained, the next step is the pumping system dimensioning. For that purpose, aspects such as the optimal number and size of the pumps, the kind, the optimal method of operation, among other optimisation issues must be solved. However, this step is beyond the reach of this investigation, since it requires a comprehensive research work by itself. Nevertheless, the results obtained from the application of the developed methodology evidence that pumping systems usually, though not always, supply an excess of energy that can impact negatively in the operating costs. In that way, pumping systems operated by using the optimal SCs can achieve a theoretical saving up 12% annual. This as long as pumps do not need to be replaced and reach a minimum efficiency expected. The optimisation methodology also provides information about the importance order of the different pumping systems in regards the cost and energy. This is, which ones of the pumping stations represent bigger savings and must supply more water to the network, and which ones are less important or are not required. Besides, regarding existing pumping systems, what pumping stations are either oversized or undersized can be known. On the other hand, the method of optimisation proves that optimal flow distribution is a complex problem that cannot be inferred at first sight. That is, better pumping conditions (e.g. low energy tariffs, higher pumping efficiencies) do not always mean cheaper pumping costs. This is because pumping heads and flow distributions can change those conditions. Additionally, some results show the possible utility of the methodology to optimise the location and use of the network storage infrastructure.

Resumen

La optimización de los sistemas de bombeo involucra tanto a los costos de inversión como los costos de operación. Dichos costos se encuentran estrechamente relacionados y se fundamentan en tres aspectos: a) dimensionamiento óptimo, b) selección óptima y c) operación óptima. Sin embargo, antes de que se pueda abordar cualquiera de los aspectos mencionados es necesario obtener la curva característica de la red. La curva característica suele asociarse con la curva resistente (CR), la cual hace referencia al caudal y altura de presión necesarios en cada estación de bombeo para satisfacer la demanda de caudal de los usuarios de la red. Para ello se debe vencer la resistencia del sistema (pérdidas de carga, diferencias de altura, etc.) que se deriva de la variación espacial y temporal de la demanda, así como de la ubicación de los puntos de descarga. La variación de la demanda se trasmite a la red mediante elementos que generan resistencia y que son operados por el usuario, es decir, válvulas, grifos, etc. En este sentido, la resistencia generada por el usuario se ajusta a las necesidades de caudal y presión en cada punto de consumo. La dificultad de determinar la variación de la resistencia generada por el usuario hace que el cálculo de las curvas resistentes y de todos los puntos que las conforman sea difícil de lograr.

Teóricamente, los sistemas de bombeo se diseñan, seleccionan y optimizan en función de los puntos de operación obtenidos por la intersección de tres curvas: a) la CR de la demanda máxima, b) CR de la demanda mínima y, c) la curva motriz del sistema de bombeo. Sin embargo, lo que se suele hacer es proponer un sistema de bombeo para luego obtener sus puntos de operación respecto de las condiciones de demanda y presión de la red. Dicho proceso ha sido ampliamente estudiado mediante la aplicación de una gran variedad de modelos matemáticos de optimización: clásicos (lineales, no lineales, dinámicos, cuadráticos, estocásticos, etc.) y metaheurísticos (algoritmos evolutivos, colonias de hormigas, “simulated annealing”, etc.). No obstante, el problema radica en que los sistemas de bombeo son diseñados tomando como referencia el punto de operación crítico (máxima demanda y máxima altura de presión), pero se optimizan dejando de lado el hecho de que para demandas menores a la máxima la altura de bombeo necesaria también es menor. Por lo tanto, no se cuantifica el exceso de energía del bombeo en relación con la mínima realmente requerida. En este contexto, se puede dar un incremento innecesario de los costos de operación que puede ser determinante al momento de escoger un sistema de bombeo u otro diferente. Por lo tanto, aunque los métodos tradicionales optimizan la operación de los sistemas de bombeo, el proceso en sí mismo se encuentra incompleto ya que los requerimientos reales de la red no se encuentran definidos apropiadamente. La solución al problema mencionado no solo facilitaría una mejor estimación y optimización de los costos de operación de los sistemas

de bombeo, sino que también conduciría a un mejor dimensionamiento y selección de las bombas que lo conforman.

El problema se puede abordar de forma diferente mediante el uso de la curva de consigna (CC). La CC es otro tipo de curva característica poco estudiada hasta ahora y de la cual se pueden calcular todos sus puntos con relativa facilidad. Esta indica el caudal y altura de presión requeridos en las estaciones de bombeo para fijar la mínima requerida en el nudo crítico de la red. Por tanto, siempre que se mantenga la presión mínima en el nudo crítico también se cumplirá con los requerimientos de presión de los demás nudos. Esta nueva perspectiva forma parte fundamental de la presente tesis. Así, el objetivo de este trabajo consiste en la formulación de una metodología para la optimización del uso de la energía y de los costos de operación de sistemas de bombeo en redes de distribución de agua.

Hasta ahora, el proceso para el cálculo de la CC se ha limitado a redes con dos estaciones de bombeo, distribuciones de caudal fijas, consumos no dependientes de la presión y sin tanques de almacenamiento. Por lo tanto, en este trabajo se amplía la metodología de cálculo de la CC a los casos mencionados. Usualmente, la CC se usa para optimizar la operación de los sistemas de bombeo ajustando la curva motriz lo más cerca posible de la CC. Sin embargo, cuando cambia la distribución de caudales de suministro entre las estaciones de bombeo cambian también las curvas de consigna además de los requerimientos de energía en cada estación. De esta forma, el presente trabajo estudia la manera de obtener la distribución de caudales óptima que minimice los requerimientos de energía respecto de la variación de la demanda y que permita converger hacia la CC óptima de cada estación de bombeo. La metodología de optimización se formula para redes con múltiples estaciones de bombeo, consumos dependientes y no dependientes de la presión, sin tanques de almacenamiento.

Para obtener la distribución óptima de caudales se proponen dos métodos, uno discreto (M-D) y otro continuo (M-C). En el M-D, la distribución de caudales se trata como variable discreta y se requiere de la formulación de conjunto finito de distribuciones previa su aplicación. En este contexto, mediante una función objetivo se evalúan todas las posibles distribuciones de caudal para cada demanda. Al final se obtienen los valores de la energía requerida por cada estación de bombeo en función de la distribución y de la demanda. Con estos valores se construyen curvas de energía en las cuales el mínimo valor indica la distribución óptima de caudal para un valor específico de la demanda. Por otro lado, cuando se obtiene la distribución óptima se obtiene además un punto de la CC por cada estación de bombeo que corresponde al caudal y altura de presión con que se da la distribución óptima. En el caso del M-C, la distribución de caudal se asume como una variable continua, por lo tanto, no es necesario obtener curvas de energía como en el método discreto. Esto se debe a que la distribución óptima viene dada por la aplicación de algoritmos de búsqueda directa. Los algoritmos utilizados son: Hooke-Jeeves, y

Nelder-Mead. De esta forma, el M-C es más preciso que el M-D y permite resolver problemas con un mayor número de dimensiones.

El siguiente paso en la investigación consiste en el estudio de la influencia de las tarifas de energía y otros costos relevantes en la distribución óptima de caudales. Para lo cual, se parte del M-C desarrollado para la optimización energética y se incluyen en la función objetivo los costos de bombeo y producción de agua. Esto permite obtener la distribución de caudales óptimas respecto del costo de la energía usada en cada estación de bombeo además de las curvas de consigna de menor costo. Al igual que en la optimización energética, la metodología está dirigida a redes con múltiples estaciones de bombeo, consumos dependientes y no dependientes de la presión y, sin capacidad de almacenamiento.

En la parte final de este trabajo se incluyen los tanques de almacenamiento dentro del proceso de optimización y cálculo de las curvas de consigna de menor costo. Esta consideración implica modificar la metodología de cálculo de las curvas de consigna. Para hacerlo, se incluyen dos costos de penalización en la misma función objetivo que se usa en el caso de redes sin tanques. De esa forma se penaliza el incumplimiento de presiones y volúmenes de almacenamiento. La optimización de realiza mediante el uso de los algoritmos evolutivos “Differential Evolution” y el “Hybrid Algorithm”. Se debe observar que en los casos de optimización previos (energía y costos en redes sin tanques), no se incluyó la penalización por incumplimiento de la presión. La razón es que el proceso de cálculo de la curva de consigna garantiza de forma implícita que la presión mínima se mantenga fija en el nudo crítico. Sin embargo, cuando se consideran los tanques, la presión del nudo crítico no puede mantenerse constante durante todo el periodo de simulación. De hecho, la presión variará en función de la variación de los niveles de los tanques y de si estos se están llenando o vaciando. Por lo tanto, para el cálculo de la curva de consigna se persigue mantener una presión igual o mayor a la mínima requerida siempre tendiendo al menor valor posible. La penalización por incumplimiento de los volúmenes de almacenamiento se considera siempre que al final del periodo de simulación los niveles de almacenamiento estén por debajo de los niveles iniciales. Se debe mencionar que los costos de penalización no son fijos, sino que son proporcionales al incumplimiento de las condiciones requeridas, lo que facilita a los algoritmos la búsqueda de la solución óptima.

Cabe señalar que no se consideran los costos de inversión en el caso de que sistemas de bombeo existentes deban ser reemplazados. Por lo tanto, este trabajo está dirigido para la optimización de sistemas de bombeo diseñados desde cero. No obstante, en el caso se sistemas preexistentes, las curvas de consigna óptimas serán útiles siempre y cuando puedan ser fijadas como políticas de operación y se cumpla con un predeterminado rendimiento en las bombas.

Para la aplicación de las metodologías desarrolladas se estudian cinco redes de distribución bajo diferentes condiciones de funcionamiento: TF, Catinen, COPLACA, Anytown y Richmond. Las primeras tres son redes sin tanques usadas en el estudio de la optimización energética y de costos en redes sin capacidad de almacenamiento. Algunas de las condiciones estudiadas son: consumos dependientes y no dependientes de la presión, número variable de estaciones de bombeo, limitaciones de caudal, etc. Adicionalmente para la red TF se ha realizado una selección de bombas a manera demostrativa de la aplicabilidad del método de optimización. Anytown y Richmond han sido usadas para la optimización de energía y costos en el caso de redes de distribución con capacidad de almacenamiento. Algunos de los escenarios estudiados son: número variable de tanques, estaciones de rebombeo, nivel variable en la succión, etc. En ninguna de las redes se han considerado múltiples condiciones de demanda tales como, caudal contra incendios, curvas de demanda estacionales, etc. Tampoco se ha considerado el efecto en la fiabilidad debido a la remoción de estaciones de bombeo o tanques. Estos aspectos requieren mayor investigación and son complementarios al dimensionamiento de las bombas.

Una vez que se obtienen las distribuciones de caudal óptimas y las curvas de consigna, el siguiente paso conduce al dimensionamiento del sistema de bombeo. Para ello se deben resolver problemas como el número óptimo de bombas, el tipo de bombas (velocidad variable o fija), el método de regulación para su operación óptima, etc. Sin embargo, este paso se encuentra más allá de los límites de este trabajo debido principalmente a que merece un trabajo de investigación en sí mismo. No obstante, los resultados obtenidos evidencian que los sistemas de bombeo usualmente, aunque no siempre, suministran agua con un exceso de energía que puede afectar negativamente los costos de operación. De esta forma, aquellos sistemas que sean operados siguiendo las curvas de consigna óptimas pueden alcanzar ahorros anuales de hasta un 12 %. Además, la metodología proporciona información sobre las estaciones de bombeo que representan mayores ahorros frente a aquellas que son menos importantes o innecesarias. Por otro lado, es posible determinar qué estaciones se encuentra sobredimensionadas o subdimensionadas. El método ha permitido demostrar que la distribución óptima de caudales es un problema complejo que no puede inferirse a simple vista. De esta forma, mejores condiciones de bombeo (bajas tarifas de energía y altos rendimientos) no siempre significan menores costos de operación. Esto se debe a que las alturas de bombeo y distribuciones de caudal pueden cambiar esas condiciones. Finalmente, algunos resultados muestran la posible utilidad del método para optimizar tanto el uso como la ubicación de los tanques de almacenamiento.

Resum

L'optimització dels sistemes de bombament involucra tant als costos d'inversió com els costos d'operació. Aquests costos es troben estretament relacionats i es fonamenten en tres aspectes: a) dimensionament òptim, b) selecció òptima i c) operació òptima. No obstant açò, abans que es puga abordar qualsevol dels aspectes esmentats és necessari obtenir la corba característica de la xarxa. La corba característica sol associar-se amb la corba resistent (CR), la qual fa referència al cabal i altura de pressió necessaris en cada estació de bombament per a satisfer la demanda de cabal dels usuaris de la xarxa. Per a açò s'ha de vèncer la resistència del sistema (pèrdues de càrrega, diferències d'altura, etc.) que es deriva de la variació espacial i temporal de la demanda, així com de la ubicació dels punts de descàrrega. La variació de la demanda es transmet a la xarxa mitjançant elements que generen resistència i que són operats per l'usuari, és a dir, vàlvules, aixetes, etc. En aquest sentit, la resistència generada per l'usuari s'ajusta a les necessitats de cabal i pressió en cada punt de consum. La dificultat de determinar la variació de la resistència generada per l'usuari fa que el càlcul de les corbes resistents i de tots els punts que les conformen siga difícil d'aconseguir.

Teòricament, els sistemes de bombament es dissenyen, seleccionen i optimitzen en funció dels punts d'operació obtinguts per la intersecció de tres corbes: a) la CR de la demanda màxima, b) CR de la demanda mínima i, c) la corba motriu del sistema de bombament. No obstant açò, la qual cosa se sol fer és proposar un sistema de bombament per a després obtenir els seus punts d'operació respecte de les condicions de demanda i pressió de la xarxa. Aquest procés ha sigut àmpliament estudiat mitjançant l'aplicació d'una gran varietat de models matemàtics d'optimització: clàssics (lineals, no lineals, dinàmics, quadràtics, estocàstics, etc.) i metaheurísticos (algorismes evolutius, colònies de formigues, "simulated annealing", etc.). No obstant açò, el problema radica que els sistemes de bombament són dissenyats prenent com a referència el punt d'operació crític (màxima demanda i màxima altura de pressió), però s'optimitzen deixant de costat el fet que per a demandes menors a la màxima l'altura de bombament necessària també és menor. Per tant, no es quantifica l'excés d'energia del bombament en relació a la mínima realment requerida. En aquest context, es pot donar un increment innecessari dels costos d'operació que pot ser determinant al moment d'escollir un sistema de bombament o un altre diferent. Per tant, encara que els mètodes tradicionals optimitzen l'operació dels sistemes de bombament, el procés en si mateix es troba incomplet ja que els requeriments reals de la xarxa no es troben definits apropiadament. La solució al problema esmentat no solament facilitaria una millor estimació i optimització dels costos d'operació dels sistemes de bombament, sinó que també conduiria a un millor dimensionament i selecció de les bombes que ho conformen.

El problema es pot abordar de forma diferent mitjançant l'ús de la corba de consigna (CC). La CC és un altre tipus de corba característica poc estudiada fins ara i de la qual es poden calcular tots els seus punts amb relativa facilitat. Aquesta indica el cabal i altura de pressió requerits en les estacions de bombament per a fixar la mínima requerida en el nus crític de la xarxa. Per tant, sempre que es mantinga la pressió mínima en el nus crític també es complirà amb els requeriments de pressió dels altres nusos. Aquesta nova perspectiva forma part fonamental de la present tesi. Així, l'objectiu d'aquest treball consisteix en la formulació d'una metodologia per a l'optimització de l'ús de l'energia i dels costos d'operació de sistemes de bombament en xarxes de distribució d'aigua.

Fins ara, el procés per al càlcul de la CC s'ha limitat a xarxes amb dues estacions de bombament, distribucions de cabal fixes, consums no depenents de la pressió i sense tancs d'emmagatzematge. Per tant, en aquest treball s'amplia la metodologia de càlcul de la CC als casos esmentats. Usualment, la CC s'usa per a optimitzar l'operació dels sistemes de bombament ajustant la corba motriu el més a prop possible de la CC. No obstant açò, quan canvia la distribució de cabals de subministrament entre les estacions de bombament canvien també les corbes de consigna a més dels requeriments d'energia en cada estació. D'aquesta forma, el present treball estudia la manera d'obtenir la distribució de cabals òptima que minimitze els requeriments d'energia respecte de la variació de la demanda i que permeta convergir cap a la CC òptima de cada estació de bombament. La metodologia d'optimització es formula per a xarxes amb múltiples estacions de bombament, consums depenents i no depenents de la pressió, sense tancs d'emmagatzematge.

Per a obtenir la distribució òptima de cabals es proposen dos mètodes, un de discret (M-D) i un altre continu (M-C). En el M-D, la distribució de cabals es tracta com a variable discreta i es requereix de la formulació de conjunt finit de distribucions prèvia la seua aplicació. En aquest context, mitjançant una funció objectiu s'avaluen totes les possibles distribucions de cabal per a cada demanda. Al final s'obtenen els valors de l'energia requerida per cada estació de bombament en funció de la distribució i de la demanda. Amb aquests valors es construeixen corbes d'energia en les quals el mínim valor indica la distribució òptima de cabal per a un valor específic de la demanda. D'altra banda, quan s'obté la distribució òptima s'obté a més un punt de la CC per cada estació de bombament que correspon al cabal i altura de pressió amb que es dóna la distribució òptima. En el cas del M-C, la distribució de cabal s'assumeix com una variable contínua, per tant, no és necessari obtenir corbes d'energia com en el mètode discret. Açò es deu al fet que la distribució òptima ve donada per l'aplicació d'algorismes de cerca directa. Els algorismes utilitzats són: Hooke-Jeeves, i Nelder-Mead. D'aquesta forma, el M-C és més precís que el M-D i permet resoldre problemes amb un major nombre de dimensions.

El següent pas en la recerca consisteix en l'estudi de la influència de les tarifes d'energia i altres costos rellevants en la distribució òptima de cabals. Per a açò, es parteix del M-C desenvolupat per a l'optimització energètica i s'inclouen en la funció objectiu els costos de bombament i producció d'aigua. Açò permet obtenir la distribució de cabals òptimes respecte del cost de l'energia usada en cada estació de bombament a més de les corbes de consigna de menor cost. Igual que en l'optimització energètica, la metodologia està dirigida a xarxes amb múltiples estacions de bombament, consums depenents i no depenents de la pressió i, sense capacitat d'emmagatzematge.

En la part final d'aquest treball s'inclouen els tancs d'emmagatzematge dins del procés d'optimització i càlcul de les corbes de consigna de menor cost. Aquesta consideració implica modificar la metodologia de càlcul de les corbes de consigna. Per a fer-ho, s'inclouen dos costos de penalització en la mateixa funció objectiu que s'usa en el cas de xarxes sense tancs. D'aqueixa forma es penalitza l'incompliment de pressions i volums d'emmagatzematge. L'optimització de realitza mitjançant l'ús dels algorismes evolutius "Differential Evolution" i el "Hybrid Algorithm". S'ha d'observar que en els casos d'optimització previs (energia i costos en xarxes sense tancs), no es va incloure la penalització per incompliment de la pressió. La raó és que el procés de càlcul de la corba de consigna garanteix de forma implícita que la pressió mínima es mantinga fixa en el nus crític. No obstant açò, quan es consideren els tancs, la pressió del nus crític no pot mantenir-se constant durant tot el període de simulació. De fet, la pressió variarà en funció de la variació dels nivells dels tancs i de si aquests s'estan omplint o buidant. Per tant, per al càlcul de la corba de consigna es persegueix mantenir una pressió igual o major a la mínima requerida sempre tendint al menor valor possible. La penalització per incompliment dels volums d'emmagatzematge es considera sempre que al final del període de simulació els nivells d'emmagatzematge estiguen per sota dels nivells inicials. S'ha d'esmentar que els costos de penalització no són fixos, sinó que són proporcionals a l'incompliment de les condicions requerides, la qual cosa facilita als algorismes la cerca de la solució òptima.

Cal assenyalar que no es consideren els costos d'inversió en el cas que sistemes de bombament existents hagen de ser reemplaçats. Per tant, aquest treball està dirigit per a l'optimització de sistemes de bombament dissenyats des de zero. No obstant açò, en el cas se sistemes preexistents, les corbes de consigna òptimes seran útils sempre que puguen ser fixades com a polítiques d'operació i es complisca amb un predeterminat rendiment en les bombes.

Per a l'aplicació de les metodologies desenvolupades s'estudien cinc xarxes de distribució sota diferents condicions de funcionament: TF, Catinen, COPLACA, Anytown i Richmond. Les primeres tres són xarxes sense tancs usades en l'estudi de l'optimització energètica i de costos en xarxes sense capacitat d'emmagatzematge. Algunes de les condicions estudiades són: consums depenents i no depenents de la pressió, nombre variable d'estacions de bombament, limitacions de cabal, etc. Addicionalment per a la xarxa TF s'ha realitzat una selecció de bombes a manera demostrativa de l'aplicabilitat del mètode d'optimització. Anytown i Richmond han sigut usades per a l'optimització d'energia i costos en el cas de xarxes de distribució amb capacitat d'emmagatzematge.

Alguns dels escenaris estudiats són: nombre variable de tancs, estacions de re-bombe, nivell variable en la succió, etc. En cap de les xarxes s'han considerat múltiples condicions de demanda tals com, cabal contra incendis, corbes de demanda estacionals, etc. Tampoc s'ha considerat l'efecte en la fiabilitat a causa de la remoció d'estacions de bombament o tancs. Aquests aspectes requereixen major recerca i són complementaris al dimensionament de les bombes.

Una vegada que s'obtenen les distribucions de cabal òptimes i les corbes de consigna, el següent pas condueix al dimensionament del sistema de bombament. Per a açò s'han de resoldre problemes com el nombre òptim de bombes, el tipus de bombes (velocitat variable o fixa), el mètode de regulació per a la seua operació òptima, etc. No obstant açò, aquest pas es troba més enllà dels límits d'aquest treball hagut de principalment al fet que mereix un treball de recerca en si mateix. No obstant açò, els resultats obtinguts evidencien que els sistemes de bombament usualment, encara que no sempre, subministren aigua amb un excés d'energia que pot afectar negativament els costos d'operació. D'aquesta forma, aquells sistemes que siguen operats seguint les corbes de consigna òptimes poden aconseguir estalvis anuals de fins a un 12 %. A més, la metodologia proporciona informació sobre les estacions de bombament que representen majors estalvis enfront d'aquelles que són menys importants o innecessàries. D'altra banda, és possible determinar què estacions es troba sobredimensionades o subdimensionades. El mètode ha permès demostrar que la distribució òptima de cabals és un problema complex que no pot inferir-se a simple vista. D'aquesta forma, millors condicions de bombament (baixes tarifes d'energia i alts rendiments) no sempre signifiquen menors costos d'operació. Açò es deu al fet que les altures de bombament i distribucions de cabal poden canviar aqueixes condicions. Finalment, alguns resultats mostren la possible utilitat del mètode per a optimitzar tant l'ús com la ubicació dels tancs d'emmagatzematge.

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Chapter 1
Introduction

Nowadays, it is observed a marked climate change due to global warming acceleration. Thus, stop global warming is one of the most critical concerns of humankind. In this context, the energy consumption optimisation can lead to reaching this goal by the reduction of emissions of polluting gases resulting from energy production. The United States energy production is responsible for 62.6% of emissions of sulphur dioxide, 21.1% of nitrous oxide emissions, and 40% of carbon emissions [1]. This without considering other problems like negative impact on water resources, waste generation, land-use change and others. The European Council in March 2007 aimed three targets relate the climate and energy until the year 2020:

a) Reduction of greenhouse gases by 20% compared to 1990.
b) Generation of 20% of primary power using renewable resources.
c) A 20% improvement in the EU’s energy efficiency.

In October 2014, the targets accomplishment were evaluated. Although there were significant advances regarding the first two objectives, projections showed that only 10% improvement in energy efficiency could be achieved. Thus, a new framework for climate and energy objectives for 2030 was agreed:

a) A 40% cut in greenhouse gas emissions compared to 1990 levels.
b) At least a 27% share of renewable energy consumption.
c) At least 27% energy savings compared with the business-as-usual scenario.

This precedent together with the increase in energy costs points out the need for efficient use and reduction of energy consumption [2].

The EPA (United States Environmental Protection Agency) estimates that between 3% and 4% of the energy consumption at USA is due to public utilities both drinking water and wastewater. This estimation is equivalent to 56 billion kilowatts and 4 billion dollars per year. For municipalities, consumption of the services fluctuates around 30-40% of the total energy consumption. Regarding the operating costs related to drinking water distribution networks, the energy consumption costs can reach up to 40%, of which a significant part is associated with pump systems [3]. Besides, pump systems consume around 20% of the world demand for electric energy [4]. Therefore, the research and study of analysis and optimisation tools to minimise the energy consumption and operating costs from pump systems are still being entirely necessary [5], [6]. This is the case of applying the called setpoint curve or minimum energy curve, [7]–[9]. Up to now the study of the setpoint curve and its implementation are still being somewhat insignificant. Thus, the present work pretends to give an overall vision of the economic savings and other benefits as resulting from its applying.

The setpoint curve definition refers to the minimum pressure head that a pumping station must supply to deliver a specific rate of flow altogether with other pumping stations (i.e. if there are more than one pump stations in the system) while the following conditions are accomplished:

- The network demand is satisfied over the whole simulation period.
- The minimum pressure required in the network is kept at the critical node of the system (i.e. demand node with the lowest pressure head of the network over the time interval of analysis).
- The tank levels are kept within the allowable ranges.

The utility of the setpoint curve is to point out the operating points for the pumping system. Usually, instead of the setpoint curve, the resistance curve is applied for the same proposal. Though, there are significant differences between them as will be described later in the corresponding sections. Moreover, the resistance curve is also known as the system head curve (SHC). However, both the setpoint curve and resistance curve can be used as SHC. Thus, there are two SHCs that will be defined from now on as the setpoint curve (SC) and the resistance curve (RC) to avoid any kind of confusion.

Up to now, the calculation of the SC only has been studied for networks with a maximum of two pumping stations without considering storage capacity and for non-pressure-driven demands (NPDD). In that sense, this study intends to expand the knowledge related to the SC.

According to the SC concept, one of its aims is to keep the pressure needs of the network at its lowest value. Maintaining this minimum pressure head means savings from the energy optimisation point of view. On the other hand, there will be several SCs for a pumping station depending on the flow distribution among the pumping stations (i.e.

when there are more than one) as well as the variation of the tank levels over the period of simulation. In this context, it is possible to think about finding the optimal SC for each pumping station that minimises energy consumption. This optimisation problem constitutes one of the parts of the present research. Although, it is focused on networks without storage capacity.

Since there are as many SCs as flow rates distributions between the pumping stations available on the network, the solution of the problem starts with the search of the optimal flow rates distribution among pumping stations at each period of simulation. For that, two methods have been proposed, the discrete method (D-M) and the continuous method (C-M). The D-M takes the flow rate distribution as a discrete variable; hence, a set of finite solutions are explored. The C-M makes the flow rate distribution as a continuous variable. Therefore optimisation algorithms, in this case Hooke and Jeeves (H-J) [10] and Nelder and Mead (N-M) [11], are applied to find the optimal distribution.

The next step was the inclusion of the energy fares and other essential costs in the search of both the optimal SC and the optimal flow distribution from the water supply sources. Thus, another part of this research aims costs consideration and analysis of its implications.

On the other hand, and as was mentioned above, the storage capacity of the network was left aside temporarily at both the energy and cost optimisation. This aspect had never been addressed before in the calculation of the SC. Thus, there was the need of finding a new approach to include the tanks in the analysis. In that way, a new methodology has been formulated in order to search the optimal SC that leads at the same time to minimise the operating costs. For that, additional optimisation algorithms were applied such as the Differential Evolution (DE) algorithm [12] or the Hybrid Algorithm (HA). Though the first one is well known within the optimisation world, the second one is an additional contribution of this research as result of the need of optimising the search-times of the DE algorithm. XºThus, the last part of the study includes the tanks availability of the networks and a new search algorithm.

In summary, there are three main parts derived from the research, the energy optimisation for networks without storage tanks, the cost optimisation without considering storage availability and the cost optimisation for networks with tanks. These three methodologies have been tested using five different networks. The networks, which have been studied under different work conditions are TF [7], Catinen [7], COPLACA [13], Anytown [14] and Richmond [15]. Networks TF, Catinen and COPLACA have been used specifically to illustrate the sections about energy and cost optimisation in networks without storage capacity. The information of the networks is presented in the corresponding headlands. Besides, in the case of TF network and by using the optimal SCs obtained, a pump selection has been done. This a demonstrative example of the application of the methodology developed. The other networks, Anytown and Richmond, are networks intensely studied within pump scheduling optimisation world. They have been applied to demonstrate the cost optimisation in networks with storage capacity. In general terms, the networks refer to the next cases of study:

- TF, Catinen and COPLACA networks have been studied as networks with a variable number of pumping stations and no storage capacity. The optimisation has been carried out for pressure-driven demands (PDM) and non-pressure-driven demands (NPDM).
- Anytown and Richmond networks have been studied as networks with several pumping stations, NPDM, and with storage capacity. Besides, in the case of Richmond network, additional features as booster-pumping stations and water sources of a variable level at the suction point are considered.

As additional information, this study can be though as the first part of a most comprehensive study, which is formed by three parts:

a) The calculation of the optimal flow distribution and the optimal SCs through the energy and cost optimisation of pumping systems of water networks.
b) The selection of the pumps that fit the optimal SCs with the minimum inversion cost. For that purpose, the number of pumps as well as several operation methods (i.e. variable speed pumps, fixed speed pumps, by-pass use, control valves, etc), are considered. Thus, the real saving calculation is achieved.
c) The re-calculation of the optimal flow distribution and optimal SCs. This by using the resulting costs of pumps selection. This is the join of the two previous parts.

Thus, the full picture shows a complex problem large enough to be addressed in parts. In that context, it must be highlighted that this work aims to solve only the phase a) of the whole issue. Though a demonstrative example of phase b) is also presented in one of the networks studied but without going into the subject in depth.

1.1. Hypothesis, objectives and assumptions

The following sections guide the research work done and, set the base for the final conclusions.

1.1.1. Hypotheses

According to the scientific method, the generation of knowledge or correction and integration of previous knowledge involves the formulation of hypothesis based on the observation. These hypotheses have to be contrasted through experimentation. In that sense, this research is based on the following statements.

a) The optimal flow distributions of pumping stations can be found by searching the optimal SC of each one of them.
b) There is an optimal SC for each pump station of a network.
c) The SC allows determining the minimum pressure heads and flow rates required for each supply source or pumping station to maintain the minimum pressure needed for the system.
d) The SC concept is suitable to carry out an energy and cost optimisation of pumping stations.
e) The minimum energy consumption of pumping stations coincides with the optimum flow distribution among them as long as the flow rate demand and minimum pressure head of network are satisfied.
f) The SC can be applied to networks with storage capacity.
g) The longer pumping time is given for the hours with lower energy costs.
h) The SC optimisation allows obtaining the optimum elevation of the tanks that lead to a lower energy cost of pumping.
i) The search for the optimal SC allows identifying those installed pumping systems that are oversized or undersized in water networks.
j) Pumping cost optimisation influences the pumping energy optimisation regarding the optimal flow distribution and minimum pumping heads required.

1.1.2. Objectives

It has already been introduced some of the advantages of the SC concept. Mainly they are two, the minimum pressure head at the critical node is kept over the simulation period, and the network demand is satisfied with the minimum energy. In addition, the hypotheses of the problem that are going to be contrasted have also been presented. In that context the overall objective of the research is stated:

"Optimising the energy and operating costs in water distribution networks with several pumping stations and storage tanks by using the SC concept".

In order to prove the validity of the hypothesis and accomplish with the primary objective of the research the following specific goals are formulated:

- To propose a methodology for the computation of the optimal flow distribution and optimal SC of pumping stations in water networks with no storage capacity and only from the energy optimisation point of view.
- To study the influence of PDD and NPDD in the search for the optimal SC.
- To formulate a methodology for computig the least-cost SC and to optimise the energy costs at pumping stations for networks with no storage tanks.
- To search for the optimal SC of pumping stations from the energy and cost optimisation in networks with storage capacity.

- To develop a computer tool that allows applying the proposed methodology by linking the optimisation problem and the hydraulic model.
- To test the methodology of energy and cost optimisation in different water distribution networks (WDNs), academic and real cases.
- To discuss the got results and build the base for future research works.

1.1.3. Assumptions and simplifications

To solve any problem and make its study easier some assumptions and simplifications have to be done. Although all the assumptions and simplifications are mentioned throughout the development of the document, it is essential to take them into account from the beginning of it. Therefore, they are presented below:

a) The mathematical model of the behaviour of the network is available.
b) The minimum pressure head restrictions are satisfied over the whole simulation period.
c) All water supply sources have pumping stations associated to it.
d) Pumping stations are represented as inflow nodes. In the case of pumping stations associated with water supply sources, only one node is needed per pumping station. In fact, it can be considered as the discharge node of the pumping stations. On the other hand, if it is about booster pumping station two nodes are required, the suction node and the discharge node.
e) There will always be a known pressure node in the network. In networks with storage capacity, the known pressure node will be given by the tanks. However, in networks with no storage tanks, a dummy reservoir will be used.
f) Dimensioning and selection of pumps do not form part of the goals of this research. In fact, this action can be considered the next step beyond of this study. Thus, curves of the pumps (i.e. performance and efficiency curves) are not known and are not needed.
g) Since pumps sizing and selection is not addressed, capital costs are not included in the cost optimisation sections. Therefore, this work is limited mostly to the analysis of new pumping systems. It may be also applicable to existing systems as long as they can be operated over the optimal SCs calculated without the need of any additional investment. Otherwise, the computed savings will have to be re-evaluated by including those additional costs to find out the real ones. However, in any case, the method will be suitable to show the optimal operating conditions of the pumping systems.
h) A SC is obtained for each pumping station available in the system.
i) When there are no tanks in the network, the hydraulic model will be static. Otherwise, it will be dynamic.
j) In the case of the cost optimisation energy fares are known and have an hourly structure.

k) When tanks are considered, the restrictions of volume storage must be met.
l) Tanks elevation is high enough to meet the minimum pressure head required in the network.
m) The cases study does not consider multiple operational conditions (i.e. firefighting flows, station demand curves, etc.). However, they can be simulated in order to get additional operating points of the pumping systems. These points must be taken into account when pumps will be selected. As was mentioned earlier pumps selection is not part of this research. In that sense, the comparison of optimisation methodologies cannot be done between water networks which already account with selected pumps.

1.2. Organisation of the document

This document is organised according to the chronological development of the research. Thus, in addition to the introductory chapter, the remaining sections of the text are listed below:

Chapter 2 addresses state of the art. Here a review of the different and more relevant pumping cost optimisation methods carried out to date is made. Besides a description of the theoretical concepts applied to the research is done. Those concepts comprehend types of hydraulic models, optimisation algorithms and the two system curves to characterise a WDN. These curves are the RC and the SC. Both kinds of curves are analysed deeply and the differences between them are discussed. The optimisation algorithms studied are H-J [10], N-M [11], and DE [12].

Chapter 3 presents the methodologies for the calculation process of the SCs. Thus, a study of the SC calculation in networks with one or more pumping stations as well as for PDD and NPDD is presented. Besides, the problems associated with the flow distribution among pumping stations are examined.

Chapter 4 exposes the new methodology developed to carry out an energy pumping optimisation. Thus, in this section the way to find the optimal flow distribution among pumping stations through two methods (the D-M and C-M) is studied. The discrete approach considers the optimal flow distribution as a discrete variable. Moreover, the results of this method have been used to validate the outputs of the C-M. The continuous approach takes the optimal flow distribution as a continuous variable and performs the optimisation by mean of optimisation algorithms. Besides, the convergence towards the optimal SC of each pumping station starting from energy curves of each one of them is discussed. This chapter is focused on networks with no storage capacity. In that sense, the energy optimisation methodology has been applied to the networks TF, Catinen and COPLACA. These networks have been studied under different work conditions such as a variable number of pumping stations, PDD, NPDD, and flow rate limitations.

Chapter 5 explains the methodology used to find the least-cost SC to reduce the operating costs of pumping systems. Furthermore, the influence of aspects like hourly energy tariffs, expected efficiency, and treatment costs, over the optimal SC is studied. As it was done in chapter 4, the methodology aims to networks with no storage capacity and can be applied to PDD and NPDD. The networks of study are TF and COPLACA. Besides in the case of TF network also a demonstrative pump selection is presented.

Chapter 6 refers to the final part of the investigation, where a reformulation of both methodologies (i.e. energy and cost optimisation) is carried out to include the storage capacity of the tanks within the optimisation process. Also, a new search algorithm, the Hybrid Algorithm is presented. This is a memetic algorithm where the local search is made by H-J algorithm and the global search by mean of the DE algorithm. This section contains the study of the Anytown and Richmond networks. These networks allow the analysis of the optimisation methodology when there are several tanks and several pumping stations.

Chapter 7 comprehends the conclusions got from the different methodologies developed in this research, i.e. energy and cost optimisation carried out in networks with no storage and considering storage availability. Also, some considerations related to the optimisation algorithms applied are highlighted. Besides, the observations from the study of the five demonstrative networks are also exposed. Finally, the quality indexes of the research are stated. The indexes point out the scientific environments where the main ideas of this study have been presented and discussed.

To end up the document, the section of *References* has been added. This section corresponds to the different sources of information related to this research.

1.3. Description of the variables

In an attempt to clarify the use of the notation of the variables Table 1 has been developed.

Table 1. Description of variables

Variables	Description
C	Emitter coefficient
Q	Leakage Flow
P	Average zone pressure
α	Emitter exponent
ND	Number of dimensions
E	Stop control value

Variables	Description
D	Step length of Hooke and Jeeves algorithm
R	Parameter of the stop criterion (Hooke and Jeeves algorithm)
α_F	Acceleration factor of Hooke and Jeeves algorithm
x, y	Variables of the test function
ρ	Reflection coefficient (Nelder and Mead algorithm)
χ	Expansion coefficient (Nelder and Mead algorithm)
γ_c	Contraction coefficient (Nelder and Mead algorithm)
σ	Shrink coefficient
Nps	Number of pumping stations
T	Number of time intervals
NP	Number of total population
F	weighting factor
Cr	Crossover constant
r_i	Random variable
ML	Maximum limit of times that the objective function does not improve
l_{min}	Minimum limit of the parameter *F*
f_{min}	Minimum value of the function
f_{max}	Maximum value of the function
hf	Friction head losses
ΔZ	Static lift
H_P	Pumping head
Z	Tank head
$H_0^{(RC)}$	Head of the resistance curve at point 0
$H_0^{(WDS)}$	Head of the curve of the water driven system at point 0
Q_{max}	Maximum demand
Q_{min}	Minimum demand
HGL	Hydraulic grade line
$p_{r,min}$	Minimum pressure required
γ	Water specific weight

Variables	Description
R_v	Resistance generated by the consumer
R	Resistance presented by the pipe
Q_i	Flow rate discharge of a pumping station at time *i*
$R_{v,min}$	Minimum resistance
$R_{v,max}$	Maximum resistance
$H_0^{(SC)}$	Head of the setpoint curve at point 0
Ps	Pressure head at suction node
pd	Pressure head at discharge node
TFD_i	Total flow demand of the network at time *i*
$Hd_{i,0}$	Initial arbitrary HGL elevation at dummy reservoir
PHc_i	Pressure head at critical node at time *i*
PH_{min}	Minimum pressure head required in the network
ΔPH_i	Differential of pressure at critical node between the pressure calculated and the pressure required
Hd_i	HGL elevation at discharge node at time *i*
Hd_i	HGL elevation at discharge node at time *i*
Hs_i	HGL elevation at suction node at time *i*
PH_i	Pressure head supplied by the pumping station at time i
Nst	Total number of stages
C_{HW}	Hazen-Williams coefficient
PH_{ij}	Pressure head of pumping station *j* at time *i*
Q_{ij}	Flow rate supplied by pumping station j at time I
X_{ij}	Percentage of the total flow demand to be supplied by pumping station j at time i
ΔX	Increase in the value of the percentage of demand
$Qmin_{ij}$	Minimum flow rate of pumping station j at time i
$Qmax_{ij}$	Maximum flow rate of pumping station j at time i
$PH_{i,resv,c}$	Pressure Head of the dummy reservoir over the period simulation *i* and the combination of distributions *c*
Nc	Number of combinations
$Q_{i,resv,c}$	Flow rate to be supplied by the dummy reservoir over the period *i* and combination *c*

Variables	Description
$PH_{r,c}$	Pressure head at node *r* for the combination *c*
TN	Total number of demand nodes
PEC_i	Pumping energy costs at time *i*
η_{ij}	Expected efficiency of pumping station *j* at time *i*
ET_{ij}	Energy tariff of pumping station *j* at time *i*
t_i	Pumping time at hour *i*
TWC_i	Treated water costs a time *i*
TC_{ij}	Treatment cost for each water source *j* at time *i*
W	Constant value that points out the peak flow over the whole simulation period
QMD	Average flow demand of the network
TT	Number of tanks
L_{ta}	Initial level of the tank *ta*
$Lmin_{ta}$	Minimum storage level of the tank *ta*
$Lmax_{ta}$	Maximum storage level of the tank *ta*
PPC	Pressure penalty cost
VPC	Volume penalty cost
$K_{1,i,n}$	Temporal coefficient for the pressure penalty costs
$\lambda_{1,i}$	Cost conversion factor of pressure heads
$Q_{i,n}$	Demand of node *n* at simulation period *i*
$K_{2,ta}$	Temporal coefficient for the volume penalty costs
$V_{ta,i}$	Volume that goes in or goes out of the tank ta at time *i*
t_b	Assumed pumping time value required for eliminating the volume deficit
λ_2	Cost conversion factor of tank volumes
LS	Local search activation limit

Chapter 2
State of the art

Before addressing the methodology for the energy and cost optimisation proposed in this research, an examination of different optimisation models described in the literature will be done. Also, a review of other topics of interest, as types of hydraulics models and the optimisation algorithms applied will be done. Besides a special attention has been given to the study of the system head curves in which the present study is founded.

2.1. Pumping operating optimisation models

For operating costs minimisation is essential to know the elements that are related to pumping costs and that have to be managed by the water utilities. In an overall way, pumping operating costs are given by:

a) the power supplied by the pumps,
b) the operating time of each one of them and,
c) the energy tariffs.

Energy fares depend on the commercial laws (i.e. supply and demand), and water utilities do not have any control of them. However, a correct application of the fares must be considered for the minimisation of costs. Thus, it is convenient to analyse the possible optimisation of both the power and the operating time of the pumps. Both terms are closely related and are combined at the end of the process in the optimal pumping schedule that will lead to the minimum cost.

For the optimisation of the pumping power, it is convenient to know how a pumping station is characterised. Usually, a pumping station is defined by three curves:

a) pump performance curve (head-flow),
b) efficiency curves and
c) system head curves (SHCs).

Without a doubt, the most important of the three curves are the SHCs, since sizing, selection and optimum operation of pumps are based on the computing of them. Hence, it will not be possible to reach the minimum cost or optimal operating cost if they are not known. Thus, it is imperative to find an easy method that allows calculation of the SHCs in water distribution networks (WNDs).

Nowadays, the method applied consists in proposing a pumps system. Then, a set of operational conditions is run to determine the operating points of the pumps to find the optimum pump scheduling with the minimum cost. The optimisation can be achieved by means of the optimal control models, also known as optimisation models. Ormsbee and Lansey presented an overview of those models [16]. Thus, it has been pointed out that energy costs may be reduced by decreasing either the quantity of water pumped or the pump head; by increasing pumping efficiency; by proper selection or combination of pumps; by using tanks to achieve high efficiency in pump operations; or by shifting pump operation to off-peak demand periods controlling storage levels and energy costs. Furthermore, when pump maintenance costs are taken into account as a part of operational considerations, optimisation will attempt to minimise the number of pump switches. The optimisation methods search the optimal values of some decision variables using mathematical models (linear programming, dynamic programming, or non-linear programming). In pumps scheduling problems, the optimisation approach may be done either directly or indirectly depending on the choice of the decision variables. In the former case, the decision variable is expressed as the fraction of time (i.e. the different periods where each pump will be in operation), or it could be explicit, e.g. setting a pump to run for half an hour. Then it becomes a binary variable. Therefore, the objective is to minimise the energy cost associated with the operation of each pump for each interval. In the latter case, the decision variable is expressed in terms of a substitute variable, such as a tank level or pumping station discharge. This means that the aim is to find the least-cost tank level evolution or the least-cost time distribution of pump flows (or heads). Then that solution needs to be converted to a pump operation policy. Both approaches have been used in the past to develop methodologies for optimizing different pumping systems, i.e., those with single- or multiple-pumping stations with no tanks [17], [18], those with a single tank with single- and/or multiple-pumping stations [19], [20], and those with multiple-tank and multiple-source systems [17], [21]–[24]. However, these methodologies were developed under the limitations of computational efficiency encountered.

As the computational resources advanced over time, the number of states and decision variables has increased, and new algorithms have been developed. Some of them involves mixed non-linear programming [25], genetic algorithms [26], [27], hybrid algorithms [28], fuzzy logic [29], ant colony [30], harmony-search optimisation [31], among others. Later studies have included multi-objective criteria in the search for the optimal pumping schedules [26], [29], [32], [33]. However, the inclusion of more objectives is making the problem more complex, and in some cases, they are unnecessary e.g. the minimum pumping cost can be reached by minimizing the pressure head supplied at demand nodes; hence, the need of two objectives one to minimize the costs and other to minimize the pressure would not be necessary. On the other hand, most previous methods were based on the use of fixed speed pumps, which have as a major disadvantage that they may produce pressures in a water distribution system that are significantly higher than required and could exceed specifications [34]. In those situations, the hydraulic efficiency of keeping pressures low (and consequently leakage) in the network is not addressed.

To improve hydraulic efficiency, the pumping curves should be adapted to be as close as possible to the SHC (i.e. discharge and pressure head required at every source of the network), [35]–[38]. Therefore, the use of variable-speed pumps can alleviate that problem and provide hydraulic and economic benefits by reaching a high efficiency, as demonstrated by Lamaddalena and Khila [39]. In the same way, Viholainen et al. [40] formulated a new control strategy for variable speed-controlled parallel pumps taking into account the relation between the so called preferable operating area and pump energy efficiency to reach a high-performance level. However, all these studies are based on the resistance curve (RC), which is usually difficult to determine in drinking WDNs as it depends on the demand variations both in time and space.

It should be noted that most of the optimisation models start with pre-selected pumps, meaning that once the pump discharge is known the head is calculated from the pump performance curve. Then, the objective function (OF) is used to search for the minimum cost value. In that way, the cheapest pumping configuration that meets the established requirements (economic limitations, physical limitations, others) is found. Nevertheless, in that context, it is not possible to find the maximum achievable cost saving since the existing pumps characteristics restrict the optimal solution. This means that if for the same flow rate a higher or lower pumping head is needed this will not be considered within the space of feasible solutions by the algorithm. This process is equivalent to adjust the operation of the network to the pumps system instead of the select the pumps system that the network really needs. Thus, there could be a pumps system that fits better the network operation and leads to higher savings regarding the operating costs. In this way, lower cost solutions are neglected. Therefore, a greater degree of freedom has to be added to the pumps. Thus, more cost-efficient solutions can be found when the pump limitations are removed from the formulation either partially or totally. Following on from that, it is assumed that optimal pump configurations that fit the water network are not known. To obtain a desired degree of freedom with regard to the operation of the pumps, Fernánez García et al. [41], [42] represented pumps as reservoirs (i.e. head nodes) in EPANET [43], considering the pump heads as decision variables. Their approach has been applied to irrigation networks. However, the interaction among individual reservoirs makes the behaviour and the optimisation process much more complicated.

Thus, an alternative approach would be to assume that the pumps at the water sources are supply-nodes whose flow rates are decision variables, just like is proposed in the present research. In fact, these nodes are equivalent to the discharge nodes of the pumps (i.e. pumps require a suction node and a discharge node). Since hydraulic models require to specify the flow rate demand at nodes a negative demand is specified in the case of nodes that represent pumps. Thus, after performing the simulation of the model, pressure heads required to supply the flow rate specified at each node are got. In that sense, there is only need of setting the flow distribution among the supply-nodes to satisfy the water demand of the network. In this way, the result is neither the flow nor the head a certain pump can provide, but the flow rate and pumping head required by the network. However, this is not an easy task since aspects like the minimum pressure head allowed in the network, storage capacity, energy fares, etc. have to be solved. Therefore, the present study is based on the pumping energy costs optimisation but using a concept known as a setpoint curve (SC) instead of the RC. These two types of curves deserve a complete section and will be explained in more detail in section 2.4.

2.2. Hydraulic Models

Ormsbee and Lansey [16] highlight the need for some WDNs mathematical models to assess pumping schedule policies. In that sense, they have made a chronological classification of them. Some of those models can include:

- mass balance,
- regression,
- simplified hydraulics or
- full hydraulic simulation.

In a *mass-balance model,* the flow into the system is equal to the network demand plus the rate of change in storage capacity e.g. a single tank system. The model neglects the pressure-head requirements to manage the flow into the tank and it is assumed that there is a pump combination available to achieve the desired change in storage. Besides, nodal pressure requirements are satisfied as long as the tank remains within a defined range of levels. The main advantage of this kind of models is that system response can be determined faster in comparison with simulation models. The mass-balance models are more suitable for systems where the flow is carried primarily by major pipelines, i.e. branched networks, rather than looped networks.

The *regression models* represent more appropriately the nonlinear nature of a hydraulic system through a set of nonlinear regression equations. The regression curves are got by running a calibrated simulation model under different states such as tank levels and loading conditions or by creating and relating a database from the actual operating conditions, i.e. pump head, pump discharge, tank levels, and system demands. Regression models provide a time-efficiency mechanism for evaluating the system response and also incorporate some degree of system nonlinearity. It has to be observed that regression curves and databases only possess information for a given range of demands. In the case that network demands are outside of the range of the database, the results will be wrong. Moreover, the regression models have to be sufficiently accurate (i.e. reproduce the response of the system accurately) to avoid accumulative errors. This error could generate a negative effect in optimisation algorithms that could lead to wrong results.

The *simplified network hydraulics* consists of using highly schematized systems or convert the system hydraulics into a linear type problem. This is commonly referred to as macro-model. For instance, Jowitt and Germanopoulos [23] presented a method based on linear programming where pumping operations are decoupled from the nonlinear hydraulics characteristics. For that reason, the network must be simplified in such way that analysis is developed only among pumps, valves, and tanks. This methodology takes into account pump efficiencies, the structure of the electricity tariff, the water demand pattern and the reservoir storage. However, due to the simplifications made by the method, some weaknesses can be found. For example, in the case of the demand nodes, the minimum nodal pressure could be under the minimum required for demand peak hours and minimum storage levels. Also, it can be found that sometimes is cheaper to pump water directly to the nodes than filling the tanks. Therefore, in most of the cases, the operation of the pumps is subjected to the nodal pressures. This is the reason why it is not possible to simplify the model.

The *full hydraulic simulation models* permit modelling the nonlinear dynamic capacity of the water distribution systems through solving a set of quasi-static hydraulic state equations. In water networks, the system of equations is given by the equations of mass and energy conservation.

In contrast with the other models, i.e. mass balance and regression models, the full hydraulic models are more robust since they can adapt their response to the variations of the system elements (e.g. tanks, pumps) and the spatial variations of the demand.

Considering the elasticity of the fluid and the pipe material as well as the temporal variation, the simulation models can be classified as Figure 1 (adapted from [44]) shows.

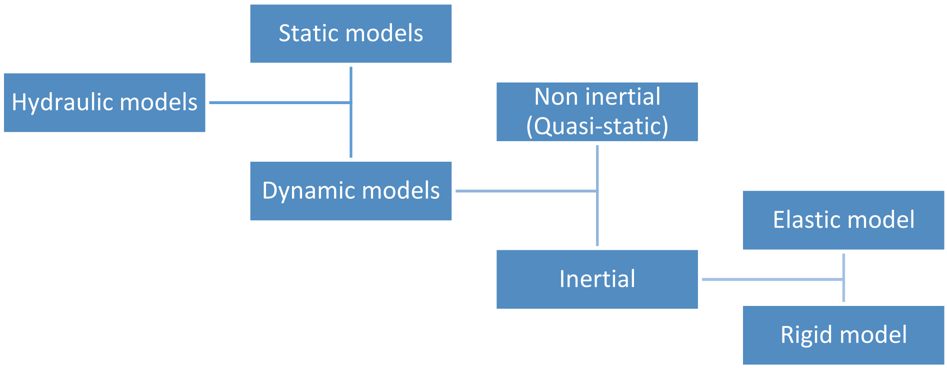

Figure 1. Types of hydraulic models

In the present research, the hydraulic models will be solved by means of EPANET software [43]. This is an analysis computer program that allows knowing the hydraulic behaviour of the different elements that can make up a distribution network, whether it is branched or meshed, from its physical characteristics. EPANET can model pipelines, driven or non-pressure driven demand nodes, tanks of various sizes and shapes, several types of valves, pumps, among other types of elements. In addition, energy analysis, water quality analysis, static model analysis, as well as extended period analysis can be performed by introducing behaviour curves or patterns to the elements of interest. It was formally developed by the US Environmental Protection Agency (EPA) and the most important results obtained from its simulations are: pressure at nodes, flow rate through pipes, water levels at tanks, the residence time of the water in the network, concentration of substances, energy consumed by pumping, as well as its costs, among many others.

Considering that EPANET software allows only two types of analysis: static and quasi-static or in an extended period, the explanation of the other models will be left aside (For a complete description of them refer [43]). A static state model can be described as the hydraulic behaviour of the network at a given instant of time. In the case of a quasi-static model what is done is to assume that the boundary conditions change slowly over time (demands, tank levels, valves, working conditions of the pumps, and several others). Under these assumptions, the effects of dynamics and inertia are considered insignificant [45], [46].

It is possible to make an analogy of the static state with a snapshot. So, an extended period analysis can be considered as a sequence of snapshots within a specific time step. The extended period simulations are commonly used to model filling and draining of tanks, opening and closing of valves, pressure and flow rates changes in response to the demand variations and automatic controls imposed by the user, among others [47].

2.3. Optimisation algorithms

Undoubtedly, the looking for optimal solutions is linked to the use of optimisation algorithms. However, there are a wide variety of algorithms that can be applied. Thus, the first issue to solve is choosing the criteria for the suitable selection of algorithms. For that purpose, the features of the optimisation problem must be defined. Earlier has been mentioned that this study consists overall of three parts:

a) Pumping energy optimisation in networks without tanks,
b) Pumping cost optimisation in networks without tanks, and
c) Pumping energy and cost optimisation in networks with tanks.

The energy optimisation is done through two methods: a discrete method (D-M) and a continuous method (C-M). These methods will be presented later in the corresponding section. However, for algorithms selection considerations, it has to be known that the assessment of the OF through the D-M does not need the use of any optimisation algorithm. In general, the D-M tests a set of solutions and chose the best of them. However, when the dimensions of the problem increase just a little (i.e. number of pumping stations), the problem becomes more complex and the amount of calculation work becomes huge. Thus, the need to implement an optimisation algorithm arises. In that sense, the C-M is based on the use of an optimisation algorithm. Besides, in the case of the C-M the decision variables are treated as continuous variables which allow more accurate in the optimisation. As the variables of the objective function (OF) are subject to the results of a water distribution network model, the function is non-differentiable. Besides, as in many problems, there are some restrictions (e.g. pressure and flow demand conditions) that must be satisfied. Therefore, the algorithm selected must be capable to address the next conditions:

- A multi-dimensional problem.
- A non-differentiable function.
- The use of continuous variables.
- The use of restrictions.

In that sense, Hooke and Jeeves (H-J) algorithm [10] has been applied. This algorithm fits with the conditions of the problem and is one of the most known and tested algorithms in the literature. On the other hand, as a way to verify the results obtained by the H-J algorithm, also Nelder and Mead (N-M) [11] was implemented. This is another known and studied algorithm that satisfies the conditions of the problem.

Regarding the cost optimisation in networks without tanks, the conditions of the problem still the same. Thus, H-J and N-M algorithms are also applied. It has to be pointed out that the cases study of networks without tanks involves the use of static hydraulic models. This means that the number of variables (i.e. dimensions) is sufficiently small to be managed by the applied algorithms. On the other side, despite both algorithms H-J and

N-M have problems with local optimum values, the use of restrictions limits the search space of the optimum solution and make them effective enough to address the issue of study.

Following with the third part of this study, when tanks are included, the number of variables of decision increase. The main reason is that the water networks models have to be solved in an extended period simulation. This means that the decision variables are not only given by the number of pumping stations but also by the number of scenarios of analysis. Even, the number can be higher if tank levels optimisation is performed. In that sense, the number of variables or dimensions of the problem is much more significant, and the search space suffer a substantial increment. Thus, the search space may be hard to delimit and problems with local optimum values become much more relevant. In that context, H-J and N-M algorithms which allow dealing with a medium-low number of dimensions and have problems with local optimum solutions are no longer suitable for the optimisation process. Thus, in order to overcome the mentioned difficulties new alternatives of algorithms were considered, such as harmony search, simulated annealing, PSO, genetic algorithms, among others. However, it has been demonstrated that genetic algorithms have a better performance against the other algorithms [48]–[51]. In that sense, the Differential evolution (DE) algorithm [12] has been implemented which is relatively easy to program and also accomplish with the conditions of the studied problem.

It is worth to point out that what is intended throgh the use of the selected algorithms is to find the best possible solutions in reasonable times. In that context the selected algorithms are reliable enough. Therefore, a comparative study of additional algorithms has not been taken into account, since the aim of the study lies on developed a new methodology of optimisation of pumping energy costs before than improve the searching times. Nevertheless, it is true that in some cases the search times may be too long. Therefore, as a contribution from this research, a hybrid algorithm has been developed, and it will be presented in the section of optimisation for networks with storage capacity. Next, a review of the applied algorithms that have been named already will be done. Also, some recommendations about the parameters of the algorithms will be given.

2.3.1. Hooke and Jeeves algorithm

2.3.1.1. Description of Hooke and Jeeves algorithm

The H-J algorithm is designed to develop a bidirectional search, that is, it advances first in the positive and then in the negative direction [10]. The search is generated for each decision variable which is part of the OF. This means that for a problem with ND dimensions, it will be needed at least $ND \cdot 2$ search directions. Moreover, the search direction advances in the direction of the variable that produces a better result for the function (i.e. when the value of the function decreases). The search process of the method is developed through the combination of exploratory moves and heuristic pattern moves (Figure 2).

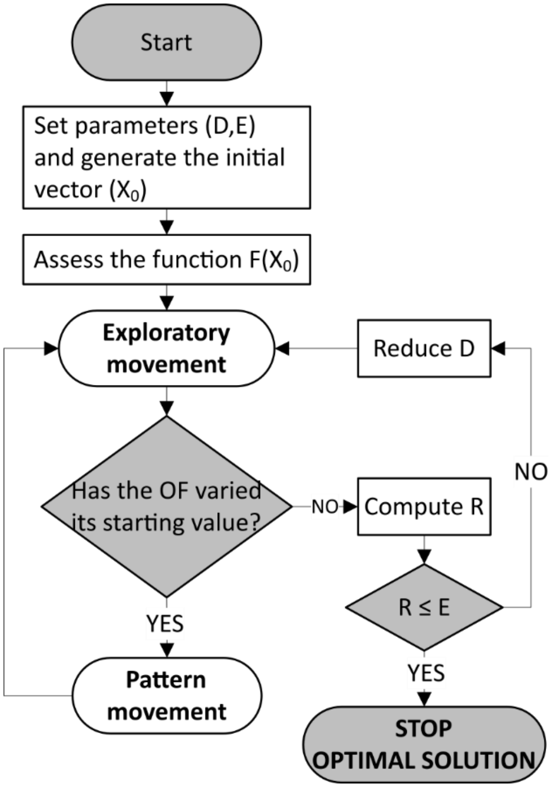

Figure 2. Scheme of the H-J algorithm

It has to be taking into acout that the parameters of the algorithm have to be pre-established before starting the optimisation (Table 2). The role of each one of them will be explained in the description of the two movements of the algorithm.

Table 2. H-J, algorithm parameters

Notation	Description of parameters
$F(X)$	OF to be minimised
ND	Number of dimensions of the function
E	Stop control value
D	Step length
R	Stop control parameter
$\vec{X}_0 = X_i, \ldots, X_{ND}$	Starting point

2.3.1.2. Exploratory movement

It is the first movement that H-J search algorithm makes. The search is carried out from an initial arbitrary vector or starting point $\vec{X}_0(ND)$ with *ND* variables or number of dimensions. In this research, the number of dimensions will be given by the number of pumping stations. Then, the OF is evaluated for the selected vector ($\vec{X}_0$) and results are recorded $F(\vec{X}_0)$.

Later, the first parameter of the method, named as step length (*D*), must be defined. This setting will be the magnitude used to modify each one of the variables of the starting vector in both directions, positive and negative. The steps to follow are the next:

1. The search begins in the positive sense; this means that the variable *i* of the starting vector is modified using the positive value of the step length (Equation 1).

$$\vec{X}_B(i) = \vec{X}_0(i) + D \tag{1}$$

2. Next, an evaluation of the OF is made $F_B(\vec{X}_B)$. Whether the result is better than the one got using only the starting point $F_B(\vec{X}_B) < F(\vec{X}_0)$, the starting search vector is updated $\vec{X}_0(i) = \vec{X}_B(i)$ and the better result is recorded $F(\vec{X}_0) = F_B(\vec{X}_B)$. Then, it is required to modify the next variable of the search vector $\vec{X}_0(i+1)$ by repeating from step 1.
3. On the opposite, if the evaluation of the OF in step 2 does not produce a better result $F_B(\vec{X}_B) > F(\vec{X}_0)$, the variable *i* of search vector is modified by using the negative value of the magnitude *D* (Equation 2), and the function is evaluated again $F_B(\vec{X}_B)$.

$$\vec{X}_B(i) = \vec{X}_0(i) - D \tag{2}$$

4. Whether the result is better than the one got using only the starting point $F_B(\vec{X}_B) < F(\vec{X}_0)$, the starting search vector is updated $\vec{X}_0(i) = \vec{X}_B(i)$ and the better result is recorded $F(\vec{X}_0) = F_B(\vec{X}_B)$. Then, next variable is modified $\vec{X}_0(i+1)$ starting from step 1. However, if a better result is not found the variable *i* returns to its initial value before adding or subtracting the *D* value. Thus, search begins with the next variable $\vec{X}_0(i+1)$ by repeating from step 1.
5. Once the search has been carried out for all the *ND* variables (i.e. *i* = *ND*), it is required to contrast the equality $F = F_B$. If the condition is accomplished, the *stop condition* has to be checked.

$$R = D \cdot ND^{0.5} \tag{3}$$

Therefore, parameter *R* must be defined (Equation 3) as well as the stop control value *(E)*. Both values *(R and E)* have to be compared. If $R > E$ the step length changes (Equation 4) and $\vec{X}_0 = \vec{X}_B$. Next, exploratory movement begins again. On the other hand, if $R \leq E$ the algorithm has found the optimal solution and search stops.

$$D = D \cdot 0.5 \tag{4}$$

Following on from the equality $F = F_B$, if both values are not equal the *pattern movement* begins. A flowchart of the exploratory movement is shown on Figure 3.

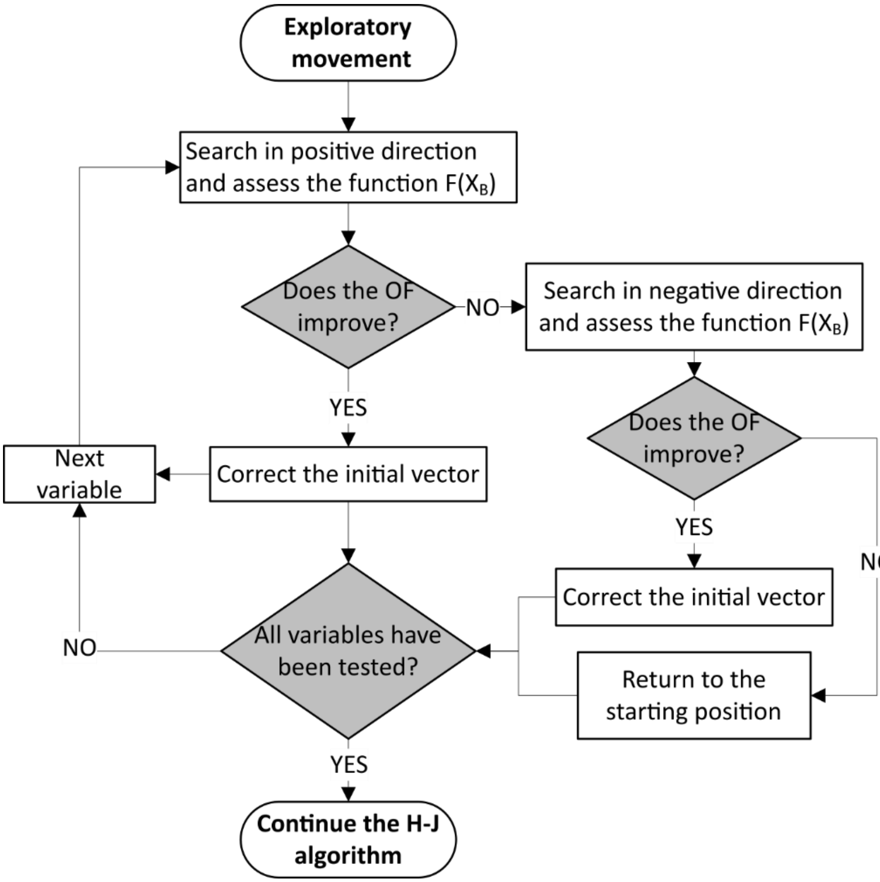

Figure 3. H-J, exploratory movement

2.3.1.3. Pattern movement

The pattern movement is developed by using the current best point of the exploratory movement $(\vec{X}_B)$ and the previous search point $(\vec{X}_0)$. Thus, those two vectors are used to make a jump in the same direction of the current best point. In order to make the jump, an acceleration factor (α) is used.

$$\overrightarrow{X} = \vec{X}_0 + \alpha_F \cdot (\vec{X}_B - \vec{X}_0) \quad (5)$$

Usually, an acceleration factor of 2 is recommended. After the jump is performed, the OF is assessed one more time $F(\overrightarrow{X})$. Depending on whether it has been got a better value of the function or not, a new search point is assumed, and the exploratory movement starts again. On the contrary, if it was not possible to get a better value of the OF, the last best point will be the new search point to start the exploratory movement. A scheme of the patter movement is shown by Figure 4.

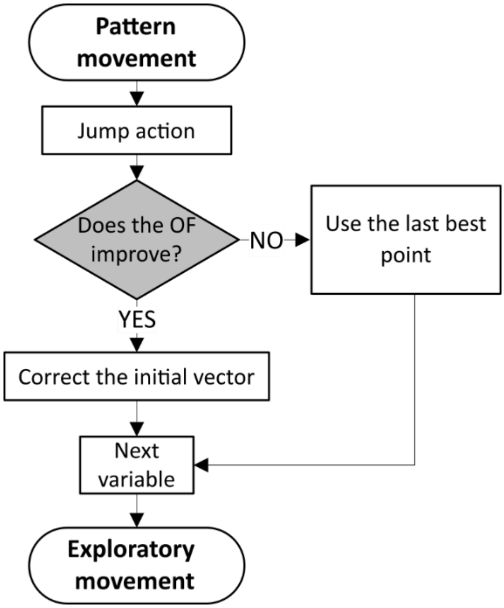

Figure 4. H-J algorithm, pattern movement

2.3.2. Nelder and Mead Algorithm

It is a heuristic multidimensional direct search algorithm [11]. The method starts making the function evaluation for the *ND+1* vertices of a simplex. In this case *ND* will be the number of dimensions of the problem. Depending on each of the values of the initial function evaluation four movements can be made: reflection, expansion, contraction, and shrink. Each movement has a characteristic parameter. Thus, the algorithm has 4 parameters that need to be adjusted. Some recommended values for these parameters are presented in Table 3, [52]. The main characteristic of the algorithm is that after each move, the simplex is rebuilt.

Table 3. Nelder Mead algorithm parameters

Notation	Description of parameters	Value
$\boldsymbol{\rho}$	Reflection coefficient	1
$\boldsymbol{\chi}$	Expansion coefficient	2
$\boldsymbol{\gamma_c}$	Contraction coefficient	0.5
$\boldsymbol{\sigma}$	Shrink coefficient	0.5

The parameters of the algorithm must satisfy the following conditions:

$$\begin{gathered} \rho > 0; \\ \chi > 1; \\ 0 < \gamma_c < 1; \\ 0 < \sigma < 1; \\ \chi > \rho \end{gathered} \tag{6}$$

The first step is to create the *ND + 1* initial vectors from where the search will start, (Equation 7). In the same way, as in the H-J algorithm, *ND* will be given by the number of decision variables, this is, the number of pumping stations in the network to be considered.

$$\overrightarrow{X_0}_i, \overrightarrow{X_0}_{i+1}, \dots, \overrightarrow{X_0}_{ND+1} \tag{7}$$

As the problem aims to minimise the function this must be assessed for each vector (i.e. vertex of the initial simplex). Then both vectors and results have to be ordered, the best value in first place and the worst value in the last place (Equation 8).

$$F\left(\overrightarrow{X_0}_1\right) \leq, F\left(\overrightarrow{X_0}_2\right) \leq, \dots, F\left(\overrightarrow{X_0}_{ND+1}\right) \tag{8}$$

Next, the reflection movement is run to rule out the worst value obtained and trying to get a better value of the function. In that sense, first of all, it is required to compute the average vector $(\overline{X_0})$ by calculating the average values of the *ND* dimensions (Equation 9) without considering the worst point located in the position *ND+1*. Later, the reflection vector is got $\left(\overrightarrow{X_R}\right)$ (Equation 10) as well as the value of the function $F\left(\overrightarrow{X_R}\right)$.

$$\overline{X_0}(i) = \frac{1}{ND} \cdot \sum_{i=1}^{ND} \overrightarrow{X_0}_i(i) \tag{9}$$

$$\overrightarrow{X_R} = \overline{X_0} + \rho \cdot \left(\overline{X_0} - \overrightarrow{X_0}_{ND+1}\right) \tag{10}$$

From the value obtained $F(\overrightarrow{X_R})$ three cases are possible (Figure 5).

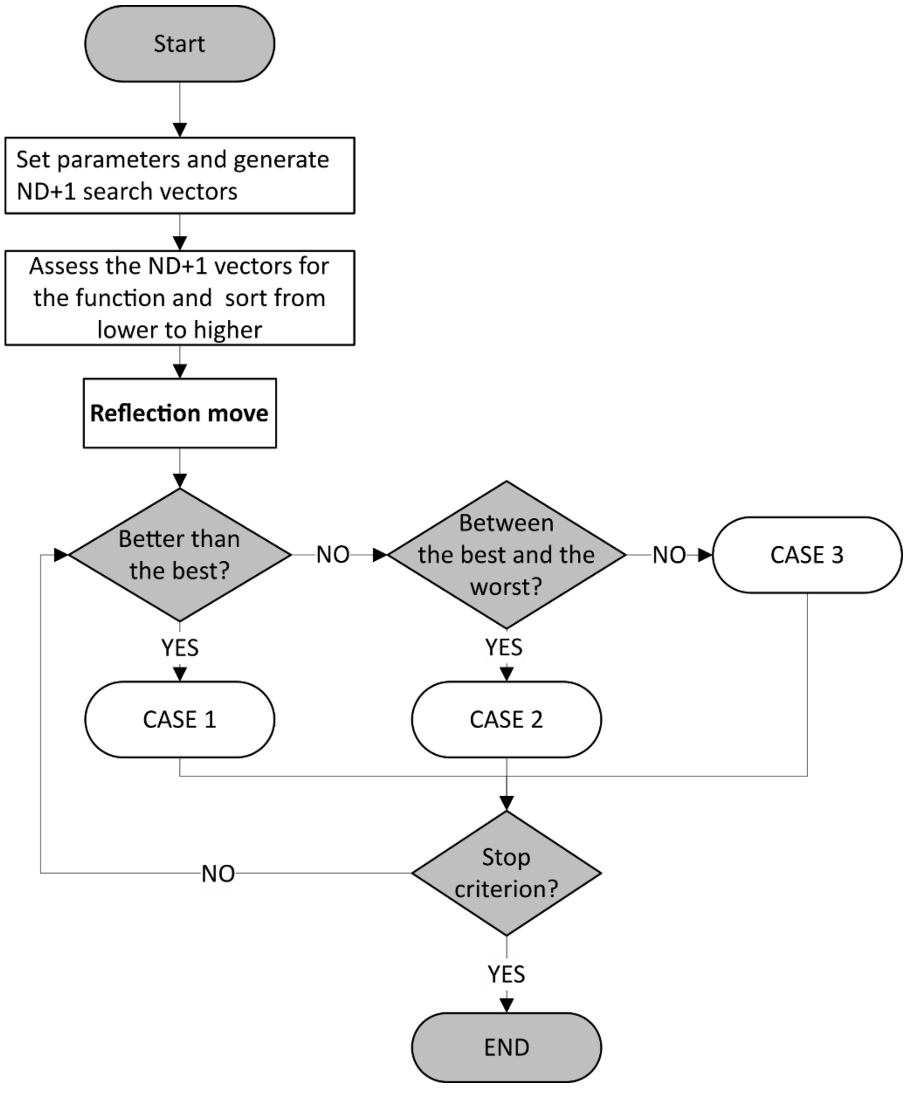

Figure 5. Scheme of the N-M algorithm

Case 1.- The reflection point improves the OF, $F(\overrightarrow{X_R}) < F(\overrightarrow{X_{0_1}})$. In this case, the expansion movement is applied trying to improve the function even more (Figure 6). Therefore, both the expansion vector $(\overrightarrow{X_E})$ (Equation 11) and the value of the function $F(\overrightarrow{X_E})$ must be found.

$$\overrightarrow{X_E} = \overline{X_0} + \chi \cdot \left(\overrightarrow{X_R} - \overline{X_0}\right) \quad \textbf{(11)}$$

- If the value of the function has improved $F\left(\overrightarrow{X_E}\right) < F\left(\overrightarrow{X_R}\right)$; a new simplex is constructed using the expansion point and by removing the worst point.
- If the value of the function has not improved $F\left(\overrightarrow{X_E}\right) \geq F\left(\overrightarrow{X_R}\right)$; the reflection vector is accepted as the new point of the simplex and the worst point is removed ending up one iteration.

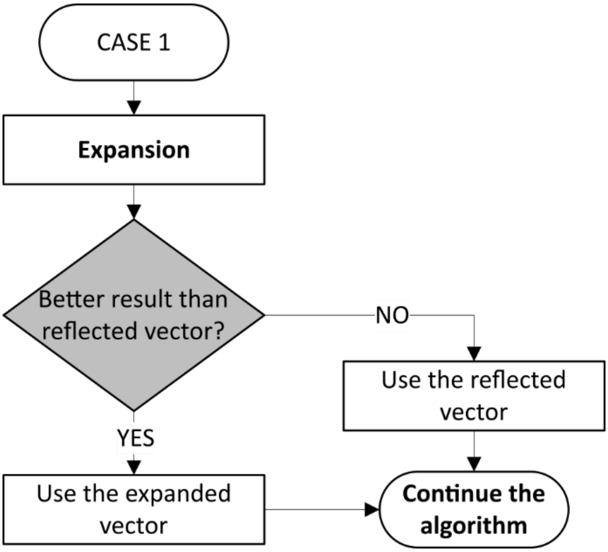

Figure 6. N-M algorithm, case 1

Case 2.- The reflection point is located between the best and the worst position $F\left(\overrightarrow{X_{0_1}}\right) \leq F\left(\overrightarrow{X_R}\right) < F\left(\overrightarrow{X_{0_{ND+1}}}\right)$. Therefore, the new better point will replace the worst ending up the iteration (Figure 7).

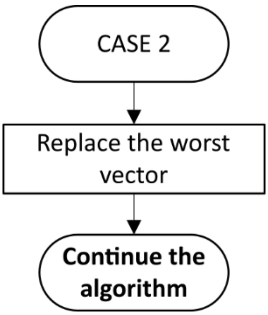

Figure 7. N-M algorithm, case 2

Case 3.- The reflection point is not better than any of the *ND* vectors $F(\overrightarrow{X_R}) \geq F(\overrightarrow{X_0}_{ND})$. This case involves two possible movements: *contraction and shrink movement* (Figure 8).

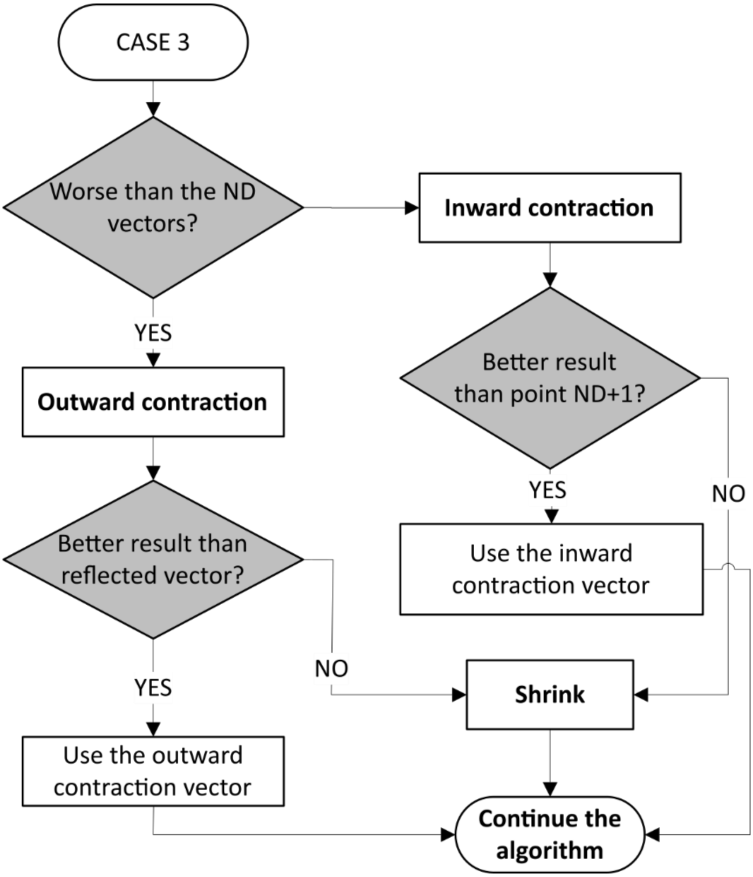

Figure 8. N-M algorithm, case 3

The contraction may be inward or outward. It is called outward if it is done in the direction of the reflected point. On the contrary, it will be contraction inward if it is done in the course of the worst position.

The outward contraction is done when the reflection vector is better than the worst point $F(\overrightarrow{X_R}) < F(\overrightarrow{X_0}_{ND+1})$. The outward contraction vector $(\overrightarrow{X_{CO}})$ is got by means of Equation (12).

$$\overrightarrow{X_{CO}} = \overline{X_0} + \gamma_c \cdot (\overrightarrow{X_R} - \overline{X_0}) \quad \textbf{(12)}$$

Then, the OF must be evaluated for the got vector. If $F(\overrightarrow{X_{CO}}) < F(\overrightarrow{X_R})$ the new point is included in the simplex and the worst point is discarded to finish the iteration. If it is not possible to improve the OF, $F(\overrightarrow{X_{CO}}) \geq F(\overrightarrow{X_R})$ then shrink movement has to be applied.

The inward contraction (Equation 13) is implemented when reflection vector produces a worse result than the *ND+1* point, $F(\overrightarrow{X_R}) \geq F(\overrightarrow{X_{0_{ND+1}}})$.

$$\overrightarrow{X_{CI}} = \overline{X_0} - \gamma_c \cdot (\overrightarrow{X_R} - \overline{X_0}) \qquad \textbf{(13)}$$

When it is gotten a better value of the function $F(\overrightarrow{X_{CI}}) < F(\overrightarrow{X_{0_{ND+1}}})$ the contraction is accepted, and a new simplex is formed. In the contrary case, the shrink movement is developed.

The shrink is made by creating *ND* new vectors (i.e. new vertices of the simplex) but without changing the best point of the initial evaluation of the function. For that, Equation (14) must be applied. In this way, new vertices of the simplex will be $\overrightarrow{X_{0_1}}, \overrightarrow{V_2}, \dots, \overrightarrow{V}_{ND+1}$. Thus, the iteration ends.

$$\overrightarrow{V_\iota} = \overrightarrow{X_{0_1}} + \sigma \cdot (\overrightarrow{X_\iota} - \overrightarrow{X_{0_1}}), \text{ with } i = 1 \dots \dots ND \qquad \textbf{(14)}$$

Each time that a new simplex is built the procedure is repeated. Therefore, a stop criterion must be added. For the cases of study, minimum squares are used. In that context, the N-M algorithm stops when the difference between the function values is lower than a pre-established error. Another stop criterion could be the number of iterations with no change of the best value. Thus, when the repetitions overcome a predefined threshold of iterations, the algorithm stops.

2.3.3. Differential evolution algorithm

In the case of non-linear and non-differentiable functions, direct search methods are the general election, for instance, the algorithms H-J and N-M which have been introduced already. This kind of methods is based on the variation of a solution vector. Then, the greedy criterion is applied. Thus, the new vector is accepted if it produces a better minimum of the OF. Though this kind of algorithms can converge very fast, they run the risk of being trapped in local minimums. This disadvantage is a big problem, especially when the search space is too broad, e.g. optimisation network models with several tanks and pumping stations and an extended period simulation. In that sense, DE algorithm [12] is a parallel stochastic direct search method designed to address non-linear and non-differentiable functions. Besides, it is capable to overcome the problem of local minimums. In the case of DE algorithm, four parameters have to be controlled (Table 4). Their values will be defined in accordance with the recommendations of some authors [12], [53]. The stop criterion will be given by a maximum limit (*ML*) for the number of times that the best value of the OF remains invariable.

Table 4. Differential evolution algorithm parameters

Notation	Description of parameters	Value
$\boldsymbol{NP}$	Population number	$10 \cdot ND$
$\boldsymbol{F}$	Weighting factor	0.5
$\boldsymbol{Cr}$	Crossover factor	0.8
$\boldsymbol{ML}$	Maximum limit	3000

In general, the optimisation problem can be stated as finding the values of the vector $\vec{X}$ to minimize the function *F(X)* as it is enunciated in Equation (15). For H-J and N-M algorithms, the number of dimensions (i.e. decision variables) of the problem is given just by the number of pumping stations. However, since DE algorithm is applied to solve models in extended period simulations, the number of dimensions is major. Thus, the number of decision variables will be given by the product between the number of pumping stations (*Nps*) and the number of time intervals (*T*).

$$Find\ \vec{X} = (x_i, \dots, x_{ND}) \text{ which minimizes } F(\vec{X}) \qquad \textbf{(15)}$$

Where, *i = 1, 2, …, ND*.

The algorithm is based on an initial population of *NP* vectors generated randomly. The value of *NP* usually will be one or twice the number of dimensions of the problem. This in the case of a problem with a significant number of dimensions. Although, it may be a higher value in cases of functions with few variables, some authors recommend a value of *NP = 10* [53].

Each element of the population ($\vec{X}_{n,g}$) will be defined as the n^{th} individual of the g^{th} generation of population. The search engine of the algorithm is defined by three sub-processes: the mutation, the crossover and the selection (Figure 9).

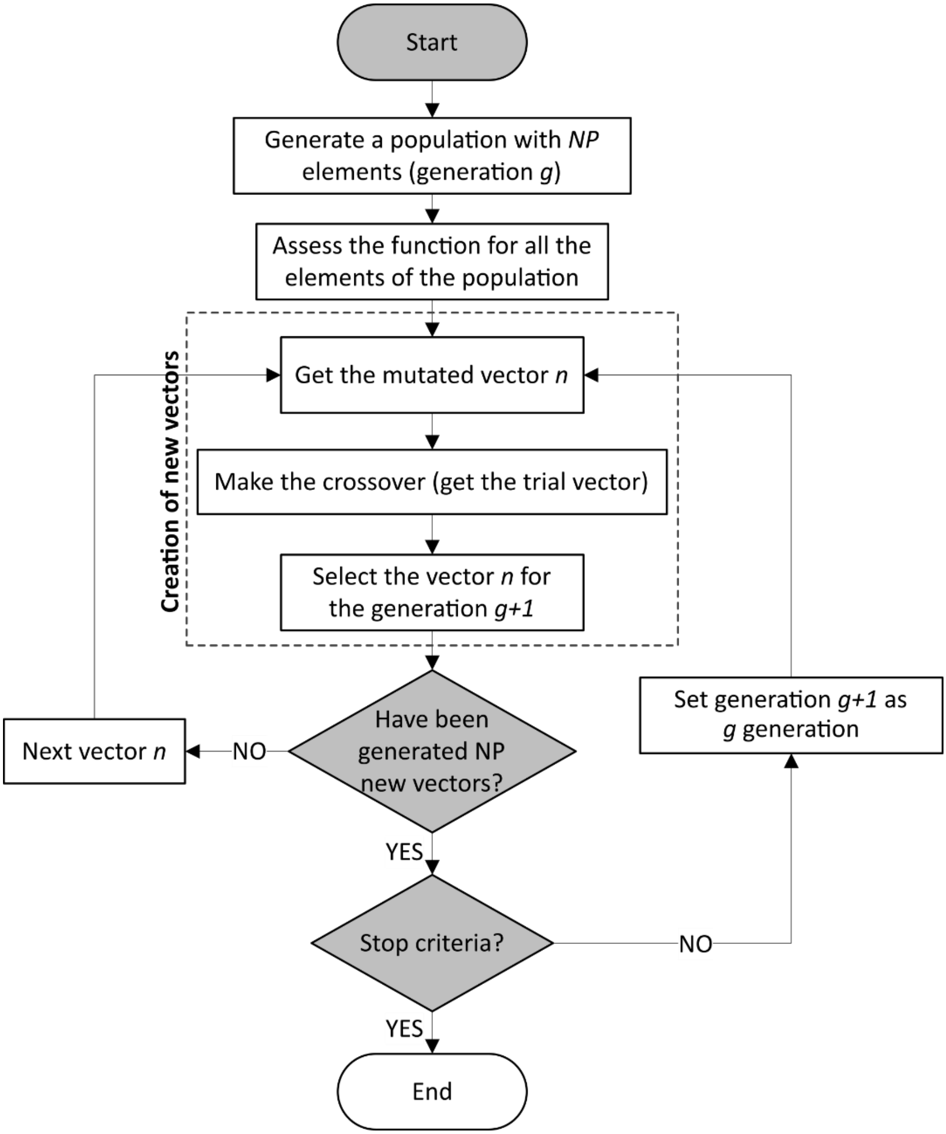

Figure 9. Scheme of DE algorithm

In the ***mutation*** step, a mutant vector $(\vec{V}_{n,g})$ is generated for each vector (n = *1,2,3, …, NP)* of the generation *(g)*. Thus, the aim of mutation is creating new parameter vectors through the sum between of the weighted difference of arbitrary vectors and another one that could be either the best, the current, or a random one. There are several equations to generate the mutation of the vector, some of them are listed next, [53], [54].

- Rand-1.

$$\vec{V}_{n,g+1} = \vec{X}_{r1,g} + F \cdot \left(\vec{X}_{r2,g} - \vec{X}_{r3,g}\right) \tag{16}$$

- Best-1.

$$\vec{V}_{n,g+1} = \vec{X}_{best} + F \cdot \left(\vec{X}_{r1,g} - \vec{X}_{r2,g}\right) \tag{17}$$

- Rand to best.

$$\vec{V}_{n,g+1} = \vec{X}_{r1,g} + F_1 \cdot \left(\vec{X}_{r2,g} - \vec{X}_{r3,g}\right) + F_2 \cdot \left(\vec{X}_{best,g} - \vec{X}_{r1,g}\right) \tag{18}$$

- Current to best.

$$\vec{V}_{n,g+1} = \vec{X}_{n,g} + F_1 \cdot \left(\vec{X}_{r2,g} - \vec{X}_{r3,g}\right) + F_2 \cdot \left(\vec{X}_{best,g} - \vec{X}_{n,g}\right) \tag{19}$$

- Rand-2.

$$\vec{V}_{n,g+1} = \vec{X}_{r1,g} + F_1 \cdot \left(\vec{X}_{r2,g} - \vec{X}_{r3,g} + \vec{X}_{r4,g} - \vec{X}_{r5,g}\right) \tag{20}$$

- Best-2.

$$\vec{V}_{n,g+1} = \vec{X}_{best,g} + F_1 \cdot \left(\vec{X}_{r2,g} - \vec{X}_{r3,g} + \vec{X}_{r4,g} - \vec{X}_{r5,g}\right) \tag{21}$$

Where,

r_1, r_2, r_3, r_4, r_5 are random indexes that indicates the position of a vector and $\epsilon \{1,2,3, \ldots, NP\}$,

F is a weighting factor ϵ [0,2]. Bibliography suggest a value of 0.5 [53].

Despite the several equations for carrying out the mutation, the Rand-1 (Equation 16) is more usual and, it will be used in the present research. In addition, the condition $n \neq r1 \neq r2 \neq r3$ must be accomplished.

Then, the trial vector that results from mixing the mutated vector with another predetermined vector, "*the target vector*", is created. This step is known as ***crossover,*** and it has the aim of increasing the diversity of the mutated vectors. The crossover or uniform crossover (Equation 22) uses a parameter called crossover constant $Cr \in [0,1]$, which has to be specified by the user. Usually is recommended a value of *Cr = 0.8* [53]. Besides, *r* is a uniformly distributed random variable $(0 \leq r < 1)$ and i_{rand} is a random index that indicates a specific position inside the vector. In that sense i_{rand} will be between the first and last position of the vector $\in 1,2, \ldots, ND$. The goal of the parameter is to assure that $\vec{U}_{n,g+1}$ gets at least one parameter from $\vec{V}_{n,g+1}$. The generation of the trial vector can be seen graphically in Figure 10.

$$\vec{U}_{n,g+1}(i) = \begin{cases} \vec{V}_{n,g+1}(i) \; if \; r \leq Cr \; or \; i = i_{rand} \\ \vec{X}_{n,g}(i) \; if \; r > Cr \; and \; i \neq n \end{cases} \quad \textbf{(22)}$$

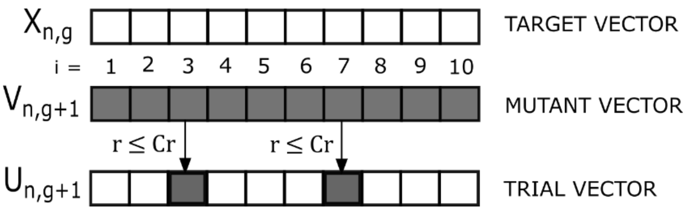

Figure 10.DE algorithm. Crossover step

Once the trial vector is got, it is time to select the new population for the next iteration. The ***selection*** consists in to choose a better individual for the population *g+1* (Equation 23).

$$\vec{X}_{n,g+1} = \begin{cases} \vec{U}_{n,g+1} \; if \; F(\vec{U}_{n,g+1}) < F(\vec{X}_{n,g}) \\ \vec{X}_{n,g} \; if \; F(\vec{U}_{n,g+1}) \geq F(\vec{X}_{n,g}) \end{cases} \quad \textbf{(23)}$$

The final solution will be given by the best individual of the population once the stop criterion has been accomplished. Though there is no any recommendation about the stop criterion, the one adopted is to allow a specific number of iterations of the algorithm where the function does not improve, then the algorithm is stopped.

The entire process can be summarised as follow [53]:

1. Select initial population (NP)
2. Assess every one element of the population $F_n(\vec{X}_{n,g})$
3. Let $\vec{V}_{n,g+1} = \vec{X}_{r1,g} + F \cdot (\vec{X}_{r2,g} - \vec{X}_{r3,g})$ where $n \neq r1 \neq r2 \neq r3$
4. Make crossover for each element of the vector $(\vec{X}_{n,g})$. If $r \leq Cr$ or $i = i_{rand}$ then $\vec{U}_{n,g+1}(i) = \vec{V}_{n,g+1}(i)$ else $\vec{U}_{n,g+1}(i) = \vec{X}_{n,g}(i)$.
5. Make the selection of the vector. If $F(\vec{U}_{n,g+1}) < F(\vec{X}_{n,g})$ then $\vec{X}_{n,g+1} = \vec{U}_{n,g+1}$ and If $F(\vec{X}_{n,g}) < F(\vec{U}_{n,g+1})$ else $\vec{X}_{n,g+1} = \vec{X}_{n,g}$
6. Repeat step 3, 4 and 5 for every one element of the population until reaching the stop criterion that has been previously established.

In an attempt to improve the time of computation of the algorithm, some authors [53] recommend only mutate the half of the population (i.e. the worst elements) and keep the better solution vectors as a measure to evolve the population better and faster. The idea is to get a lower number of evaluations of the function. Thus, a substantial saving of time can be achieved. However, it has been observed that when only the half of the population is mutated the effect is the inverse. This means, a lower number of iterations is needed but also a major number of generations has to be evaluated. On the contrary, when the original algorithm is kept, a lower number of generations have to be evaluated, though the number of iterations increases. On the other hand, the original algorithm borrows from N-M algorithm the idea of employing information from the population to alter the search space. In that context, if only the half of population is considered, only a part of the available information is included. Therefore, in the present research work, the original algorithm is applied [12].

Since DE algorithm only has three movements, the methodology is easy to program. Besides, it requires the definition of only two parameters, one related to the mutation and other related to the crossover. Actually, mutation and crossover are the most relevant steps of the algorithm, since the last one (i.e. selection) consists in knowing if the previous steps were successful. Both steps are based on a starting population big enough and diverse to achieve a suitable search of the global optimum. Otherwise, the algorithm can stagnate. Thus, the generation of the population is a process essential to the success of the optimisation. On the other hand, it has been observed that the algorithm finds quickly the space of solutions where the global optimum is. In that sense, this is a time-efficient algorithm. However, once DE has reached that space, the velocity to find the global optimum decreases substantially. Thus, this aspect of the algorithm needs to be improved.

It is important to remember that the fourth parameter of Table 4 (i.e. the ML parameter) is not related to the algorithm directly but with the stop criterion. Thus, it has to be big enough to involve all the elements of the population, this is, all population has to go across mutation, crossover and selection. Depending on its value, the time of computing will be more significant. In the cases of study that will be presented later (i.e. more complex OF) values of 1000, 3000, 5000 and 10000 has been applied. However, the chosen number will depend on the reliability of the solution. Hence, it may be advisable to use a middle value (3000 or 5000).

2.4. System head curves

As was mentioned earlier, the system head curve (SHC) is an important tool for selecting pumps and to achieve economic-effective pumping politics to minimise the operating costs. Frequently, the SHC is expressed in terms of the resistance curve. In fact, they are assumed wrongly as synonyms. However, there are two types of curves to characterise water networks:

- the resistance curve (RC), and
- the setpoint curve (SC).

Thus, it is not correct to refer to the SHC only regarding the resistance. Since the SC is the cornerstone of this research, both concepts (RC and SC) will be discussed to identify differences between them.

2.4.1. Resistance Curve

RC is based on defining the resistance or head loss of each element of the network. The flow has to overcome that resistance to be delivered at demand nodes under atmospheric pressure. In networks that do not operate by gravity, an external system that provides additional power is needed to overcome the network's resistance (i.e. a pumping system). The RC can be generated for the whole system or it can be referenced to a point. In the case of the whole system, the RC encompasses the energy requirements before and after the pumping system, i.e. the suction and discharge points. On the other side, when it is referenced to a point, the RC is only given by the energy requirements after the discharge point of the pumping system. The use of one approach or another depends on the network features. For instance, there could be more than one tank supply before the pumping system, so the formulation of the RC referenced to one point may be more convenient. Otherwise, it would be necessary to solve the tank supply system before the pumping station to find the RC of the whole system. It must be taken into account that when the RC is referenced to one point, the energy conditions previous to this point have to be considered within the curve of the water distribution system (WDS).

The amount of head required to overcome the resistance of the system is dependent on two aspects: the rate of discharge through the pumps, and the elevation differences due to system characteristics and topology. The first case involves the head loss due to friction at pipes and minor losses at pipe fittings, valves, pipe bends and others. The second one is referred to as static lift. For calculating the RC, some problems have to be taken into account. In the simplest case, a pump delivers water through a pipe from a tank to another in a higher elevation where levels are variable (Figure 11).

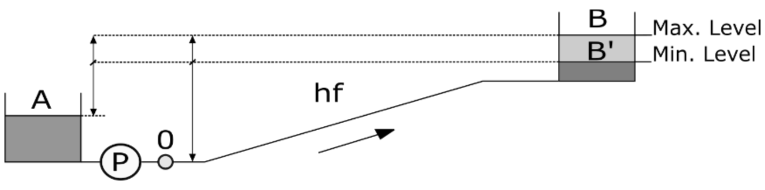

Figure 11. Network with two tanks. The discharge tank has variable levels

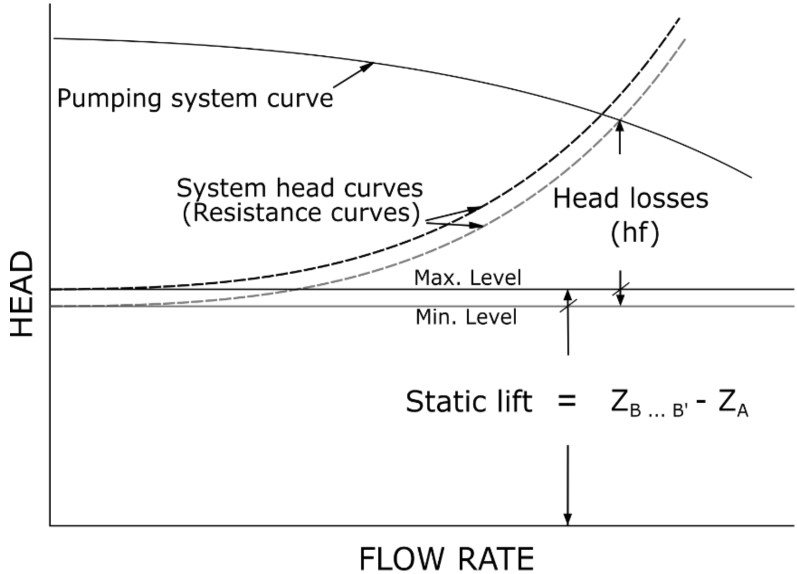

Figure 12. Resistance curves for a simple network with two tanks

In this case, the RC of the whole system (Figure 12) is got by computing the friction head lost (hf) and the static lift ($Z_B - Z_A$). However, if tank levels change the static lift will also change ($Z_{B\prime} - Z_A$) and there will be as many curves as static lift variations. It means that there is a RC for each operating condition. The pump head (H_P) required to overcome the resistance of the system will be given by:

$$H_P = Z_{B\dots B`} + hf(Q) - Z_A \qquad \textbf{(24)}$$

To simplify the RCs computing, the standard practice is to define them only for the maximum and minimum static lift and for the maximum and minimum water demand. The RC referenced at one point (point 0) is described by Equation (25). Besides, the curve of the WDS is given by the pump head and the tank head ($\mathrm{Z_A}$).

$$H_0^{(RC)} = Z_{B\dots B`} + hf(Q) \qquad \textbf{(25)}$$

$$H_0^{(WDS)} = Z_A + H_P \qquad \textbf{(26)}$$

In the case of a little more complex system (Figure 13) with more than one tank, there are three branches delivering water to three reservoirs at different elevations.

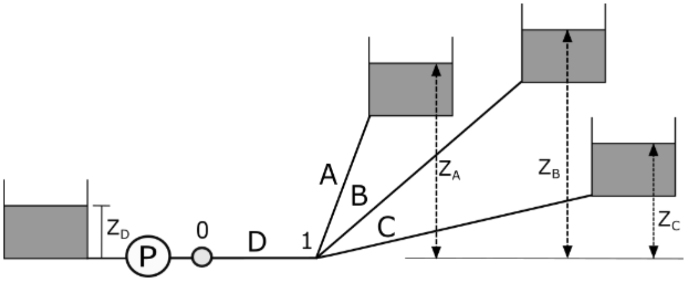

Figure 13. Network with one pumps system and three points of discharge

It has to be observed that the total flow is the sum of the branch flow, and the frictional resistance in point 1 is the same for the three branches. Therefore, the process to build the RC of the whole system can be summarized as follows:

a) the expression of the resistance of each pipe section where there is a change in flow rate demand must be defined separately,
b) the head points of each element will be computed for several flow rates,
c) the curve A+B+C will be given by the sum of flow rates that produce the same head, and
d) the RC A+B+C+D is obtained by adding algebraically the head points of pipe D and pipes A, B, and C. This is possible since pipe D is in series with pipes A, B, and C.

In this example, it has been assumed that pump is discharging to all tanks but there is a limiting liquid level elevation for each tank. When that limit is exceeded, lower-level tanks are fed by higher-level tanks and the pump. This fact has not been considered. However, the aim is to show the general process to compute the RC of system.

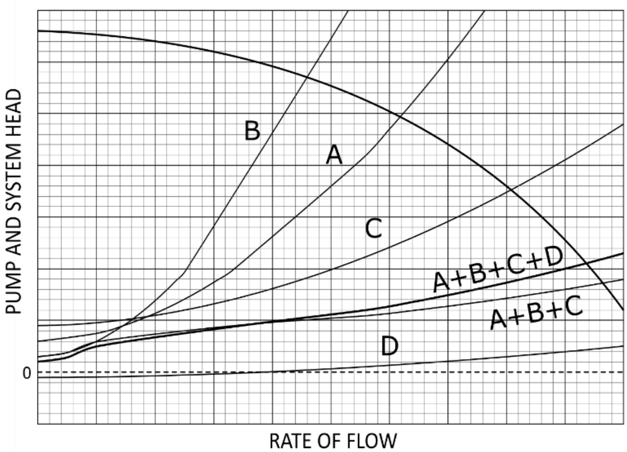

Figure 14. Resistance curves of a branched network

The resistance of each element (Figure 14) will be given by the next group of equations.

Section D

$$H_D = -Z_D + hf_D(Q_A + Q_B + Q_C) \tag{27}$$

Section A

$$H_{1\_A} = Z_A + hf_A(Q_A) \tag{28}$$

Section B

$$H_{1\_B} = Z_B + hf_B(Q_B) \tag{29}$$

Section C

$$H_{1\_C} = Z_C + hf_C(Q_C) \tag{30}$$

Now it is possible to find the resistance at point 1. The RC will be given by the adding of the flow rates that pass by each branch at a specific moment but only when all of them produce the same system head.

$$H_1 = f(Q_A + Q_B + Q_C) \tag{31}$$

For calculating the head curve of all the system, it is needed to add to the Equation (31) the head of section D given by Equation (27).

$$H_P = H_1 + H_D \tag{32}$$

In Equation (32), the flow rate that passes through section D is equal to the sum of the branch flow (A+B+C). Thus, the head of the RC is obtained adding both heads (i.e. section A+B+C and section D).

In the case that the RC is referenced to one point (i.e. point 0), most of the equations remain the same. However, the Equation (27) will be given by:

$$H_D = hf_D(Q_A + Q_B + Q_C) \tag{33}$$

Thus, the RC at point 0 will be formulated as:

$$H_0^{(RC)} = H_1 + hf_D(Q_A + Q_B + Q_C) \tag{34}$$

To include the system before the point 0 (i.e. the energy in Z_D), the curve of the WDS will be defined as:

$$H_0^{(WDS)} = Z_D + H_P \tag{35}$$

So far, a way to determine the RCs has been explained. This method could be applied to closed networks as long as they are decomposed in branched networks and the direction of the flow rate is known. However, water distribution networks (WDNs) are much more complicated and therefore finding the resistance of each element could be a rather hard task. Besides, the number of pumping stations, as well as the number of tanks and changing conditions over the day (i.e. demands variation, tank levels, different flow discharge at pumping stations and many others), turn the process in an even more difficult task.

Walski et al. [47] propose a method using a network model with a suction tank and a discharge tank analogue to the first example given before (Figure 11). The difference is that some operational conditions, as average demand, tank water levels, and others, can also be included. The RC for a set of specific operational requirements (i.e. tank suction level, discharge tank level, wells that are working, etc.) is got by breaking the model into two parts (Figure 15). The first part goes from the suction tank until the suction node of the pump. The second part goes from the discharge node of the pump through the network until the discharge tank. The pump is not specified. The flow rate to be tried is allocated to the suction node as consumption and as inflow to the discharge node. Once the model has been solved the difference of head between the two nodes will be the head needed to deliver the flow rate. Thus, one point of the RC is got. The curve is completed by testing more flow rates. Also, more curves are obtained by changing the operational conditions.

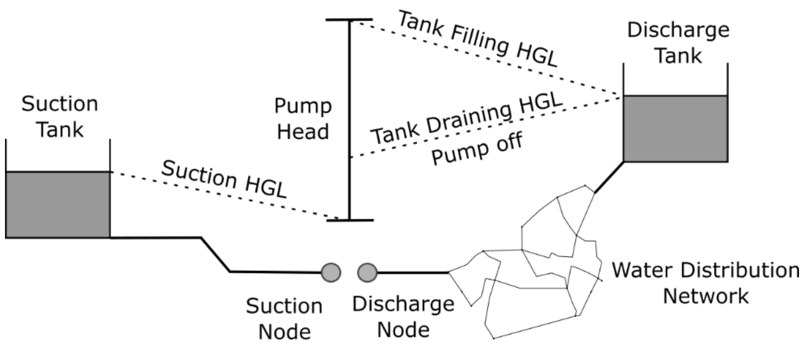

Figure 15. System head curve for a water network with a discharge tank

One of the main assumptions of the method is that the discharge tank has enough elevation to ensure the minimum pressure required in the consumption nodes when the pumping station is not running. This is not always the case, since in times of maximum demand the pressure head may be insufficient. Thus, the problem is the method considers the tank is filling all the time and do not take into account the fact that it can also supply water to the network at the same time as the pump. In that case, the RC of the system is different. Actually, without the discharge tank, there is no reference point to get the RC. Besides, the methodology only has been tested for one pumping station and one discharge tank.

In WDNs one known analogy to understand the behaviour of the RC is to consider a simple system that can be a suction tank with constant level, a pump, a conduction and a valve at the end of it (Figure 16). The valve tries to simulate the resistance created by consumers. It means when the maximum demand (Q_{max}) occur the valve is open totally and, the RC presents the minimum resistance. On the opposite when the minimum demand happens (Q_{min}), the valve is close partially and the SHC presents the maximum resistance (Figure 17).

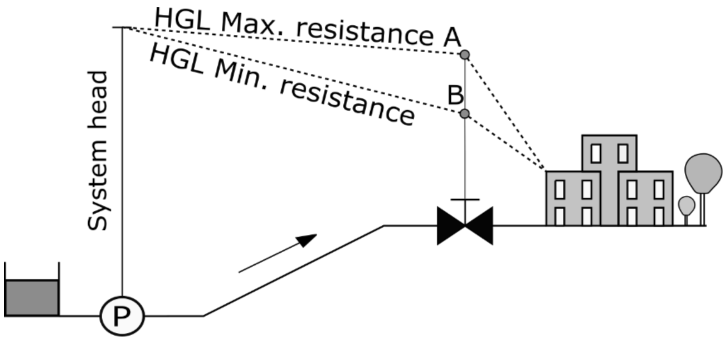

Figure 16. Simple network with a valve

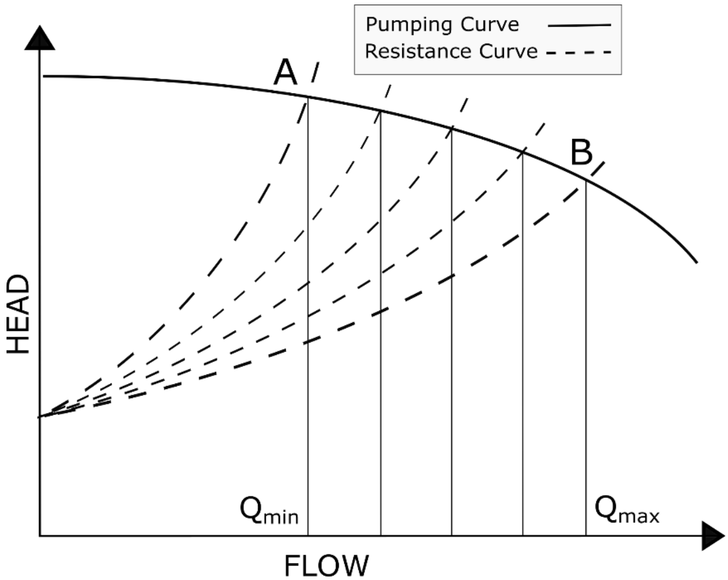

Figure 17. Resistance curves for several demands

Based on this assumption Walski [55] propose a method to calculate de RC for closed systems with pressure-driven demands (PDD). The method begins by replacing the demand by an emitter coefficient that relates the actual demand to the actual pressure head. However, the actual pressure head will depend on the installed pumping system.

Therefore, the resistance of the network will be imposed since the beginning by the existent pumps and not by the consumptions. In addition, there are some inaccuracies that are presented in the base analogy. For instance, when the network demand increases, more energy than when the demand is lower, is required (Figure 12). Thus, it is not correct to suppose that for a lower flow rate demand a bigger resistance must be overcome, as in the case of the valve. However, depending on the pressure head available, the tap of the consumer will be opened if the pressure is too low or will be closed if it is too high. Although, this kind of variations are unpredictable and cannot be simulated. Thus, a hydraulic model simplifies those variations by mean of distribution nodes where the aim is to keep a minimum pressure big enough to defeat the resistance of the inner elements at the user's edifications. Besides, it is not true that the resistance of the system is imposed by the consumer, but by the conditions of pressure head at the supply source. For instance, a pump on conditions of maximum demand will provide a much lower pressure head than on conditions of minimum demand. In that sense, when the network pressure is low, the user will open its valve at maximum creating a condition of minimum resistance. On the opposite, the user will create a maximum resistance condition when the pressure head is too high by closing his tap. However, if the pressure head available on conditions of minimum demand were the same as in the conditions of maximum demand (i.e. a low pressure), the resistance created by the consumer will be the minimum as the valve will be opened at its maximum range. This also happens when there is overpressure, i.e. either in conditions of maximum or minimum demand, the resistance generated by the operation of the consumer will be the same, in this case, the maximum. Moreover, if it is thought that in a network there are elements that have both different sizes and elevations it is not quite right to suppose that a specific flow goes through all the elements producing the same resistance in all of them, as it is done in the analogy of the valve. Thus, the RCs will not be uniform as it can be seen in the case of the network with several tanks (Figure 14).

From what has been mentioned so far, it seems like although there are several methods to define the RC, these methods are only useful for branched networks [47], [55]–[57]. Moreover, in the case of WDNs that are much more complex systems (i.e. a combination of looped and branched pipes), it is not clear yet how to find the RCs quickly. It has to be highlighted that the concept of RC might not be applicable for hydraulic network models as the resistance creating by the users is not considered.

2.4.2. Setpoint Curve

SC might be defined as a theoretical curve that point out the minimum energy (in terms of pressure head) required on source points (storage, pumping station) to meet the minimum pressure required in each demand in the network. Therefore, it is a representation of the pressure head versus flow at a given point in the system. However, the definition of the SC use to be confused with the RC where the resistance generated by consumers operation has a high importance. Moreover, for a better understanding of the SC and RC it is required to compute both curves. For do that, a simple case is applied. Thus, in Figure 18 a pump supplying water to a consumer is represented. The consumer can be represented in two different ways:

- as a demand node (Q_D) in point D where a minimum pressure ($p_{r,min}$) is required, or
- a resistive element which discharges freely to a particular level.

In fact, $p_{r,min}$ is defined as the pressure that is necessary at point D in order to guarantee that the flow through the resistance R_v is the demand Q_D. These two ways of representing consumptions is what is commonly known as time-driven analysis or pressure-driven analysis in water distribution systems analysis.

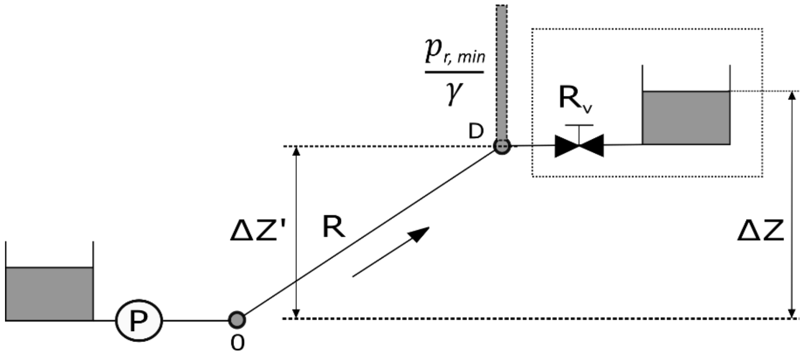

Figure 18. Supply to a consumer (D) from a pump

For computing the RC, it is assumed that the resistance R_v generated by the consumer can be determined in any moment. Thus, the head needed (H_0) at point "0" to deliver a certain flow rate is given by Equation (36).

$$H_0^{(RC)} = \Delta Z + R{Q_i}^2 + R_V{Q_i}^2 \quad \textbf{(36)}$$

Where,

ΔZ is the static lift,
R is the resistance presented by the pipe,
R_V is the resistance generated by the variation of the consumption, and
Q_i is the flow rate demand at time i.

By using Equation (36) several RCs can be generated as Figure 19 shows. Moreover, when the flow rate demanded is the maximum (Q_{max}) then the resistance is the minimum $(R_{V,min})$. Otherwise, when the flow rate is the minimum (Q_{min}) the resistance is the maximum $(R_{V,max})$. The problem of the determination of these curves appears when the R_V values of the consumers (point D) are not known. That is, we cannot perform a pressure-driven demand representation and instead, it is necessary to perform a non-pressure driven representation of the demands.

On the other hand, the minimum pressure required in the installation $(p_{r,min}/\gamma)$ is usually determined by the maximum flow conditions. Therefore, there is a relationship between the minimum pressure, the minimum resistance, and the maximum flow rate shown by Equation (37).

$$R_{V,min}{Q_{max}}^2 = \frac{p_{r,min}}{\gamma} \quad \textbf{(37)}$$

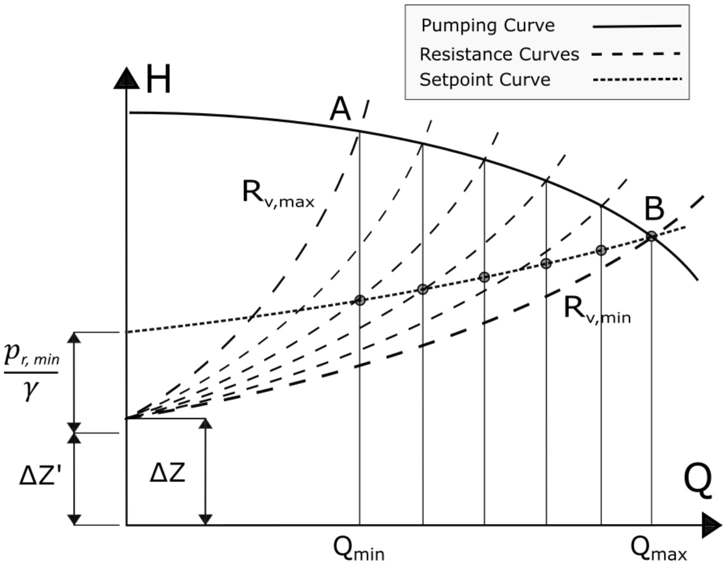

Figure 19. Supply to a consumer (D) from a pump. Resistance curve at O.

Thus, by means of Equation (36) and (37) is possible to find the point B (Figure 19) that is common for both the RC and SC. The same relation (Equation 37) can be applied to several flows. Therefore, the term of resistance generated by the user can be replaced by the minimum pressure (Equation 38). Then, the SC of the Figure 19 can be generated. In that sense, the SC can be understood as the RC that produces the minimum pressure head required regarding the water demand variation.

$$H_0^{(SC)} = \Delta Z + R{Q_i}^2 + \frac{p_{r,min}}{\gamma} \quad \textbf{(38)}$$

The SC can be defined simply as the line of flow and pressure head that a pumping station must follow to guarantee the minimum pressure required on the network. Usually, the pressure head only is checked in a reference node. The reference node is the critical node of the network (i.e. the node with the lowest pressure head). It must be considered that the critical node is not a fixed node and it could change its location depending on the variation of the network demand.

Another way to understand the SC is visualizing a simple system with a tank source, a pumps system, a pipe and a pressurised tank at the end (Figure 20). In the suction there is a *ps* pressure head and in the discharge tank a pressure head *pd* must be reached. Thus, the SC will be given by the addition of two terms, the head independent of the flow rate (i.e. static lift and desired pressure head) and the head due mainly because of the pipe friction (Figure 21).

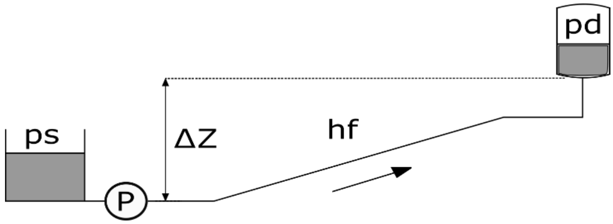

Figure 20. Network with a pressurised tank

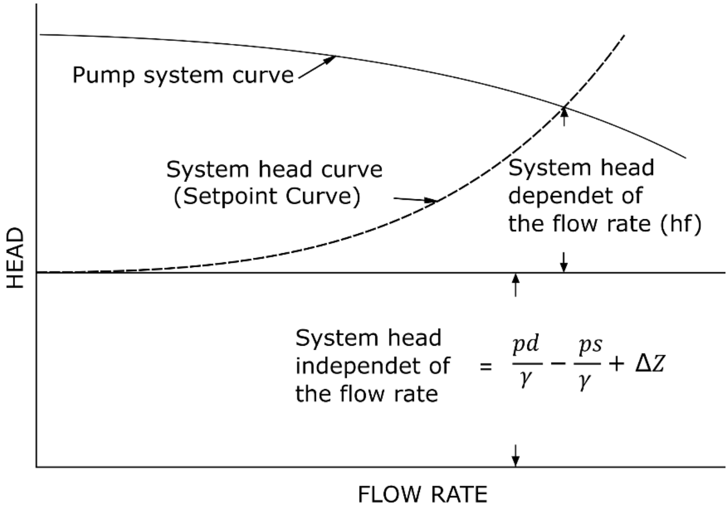

Figure 21. Setpoint curve definition

In water networks, the operation of the represented system (Figure 20) is very similar, but instead of the pressurised tank, there are consumption nodes that require a minimum pressure head condition. As it is not recommended and sometimes not possible to consider all the nodes, at least the pressure head at the critical node must be satisfied. In this way, the SC can be understood as the representation of the system head required to keep the minimum pressure head requirement at the critical node while demand is met. The big difference between the RC and SC is that SC does not try to consider the variation of the resistance generated by consumers or the valve as in the analogy (Figure 18). But, it guarantees a minimum pressure on every point upstream the final node of consumption (Figure 22).

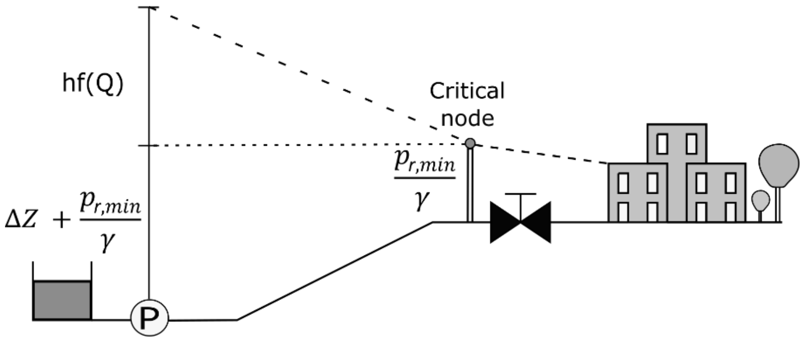

Figure 22. Setpoint curve concept

As long as suction level and minimum pressure required be kept at the critical node, there will be only just one optimal SC for each pumping station. This is independent of the number of pumping stations as it could be seen in the examples presented later. Likely, there will be several critical nodes with different elevations over the simulation. However, this aspect does not influence the number of SCs but the elevation gradient between the points of the optimal SC where the critical node changes.

In the case of networks with tanks, it is not possible to hold a constant value of the minimum pressure over the whole period of simulation. Though, the minimum pressure is guaranteed. This because of sometimes exist overpressure in the network as a condition to fill the tanks which are at higher points. Besides, tank levels change for an extended period simulation. The variation of the tank levels also affects the pressure head in the network when is the turn of tanks to supply water to the system. Thus, because of the variation both the pressure at the critical node and tank levels, the SC will have an irregular shape. This can be appreciated in the subsequent sections.

Applications for the setpoint curve

In the case of the RC, it is supposed that once the curve of resistance for both the minimum demand and maximum demand intersect the pumps system curve, it is possible to define the operating points of the system. In that context, if there is a different pumps system, the operating points will change. However, in the case of the SC, the points over the curve are the operation points of the system that meet the requirements of the network. In that way, independently of the selected pumps system, the operating points will be the same all the time. Of course, the SC can change, but also it is possible to get the optimal SCs from two different approaches that are energy and operating costs. Thus, the direct utility of the SC is the proper sizing and selection of the pump systems that fit the network needs. Therefore, the SC could be used for energy optimisation and operating costs optimisation in pumping systems. In fact, this is the aim of the exposed research.

When there are tanks in a network, if they are located too high it could be expensive to fill them. On the contrary, if they are placed in a too low elevation, they will be another consumption node without real energy saving. Since the SC is used to find the minimum energy needed at pumping stations to satisfy the pressure requirements of the network, it can also be used to find the optimal location and sizing of the tanks. In such way, tanks will contribute to the energy and cost optimisation in a WDN by reducing the head requirements of the SCs.

2.4.3. Similarities and differences between the resistance curve and the setpoint curve

Usually, a significant problem derived from the misunderstanding between of these two concepts both RC and SC is derivated, so it is worth to highlight the similarities and differences between them.

Some similarities can be summarised as follows:

- Both curves are used to define the operational points of the pumping stations.
- The two curves represent the pressure head required to deliver a specific flow rate.
- A water network model is needed to get the curve. It has to be accurate enough to represent the reality.

The main differences are:

- There will be infinite RCs depending on the behaviour of the demand as well as the configuration of the system (i.e. pipes, tank levels, valves, etc.). This means that RCs change if the resistance of the network also changes. All these curves will be delimited by the maximum and minimum RC (i.e. minimum and maximum demand). In any case, always there will be at least two to find the operating points of the pumping system. On the other hand, the SC neglects the resistance variation that depends on the consumer by always supplying the minimum pressure needed on the network according to the demand change. This means that the consumer will manage the resistance depending on the pressure head available. Thus, there will be just one SC (Figure 21). It has to be mentioned that, the SC will be different in case of variations of both the flow distribution among pumps and the pressure of the critical node. However, always it is possible to converge to just one SC. This aspect will be demonstrated in the later sections.

- Other difference is that in water networks the SC can be generated manually in a straightforward way, but the RCs need a more complicated process for their computing.
- The main difference is that the RC is based on the variability of the resistance imposed by the consumer. However, the SC is based on the assumption that the resistance of the system is imposed by the head on the supply source, always creating a condition of minimum resistance.

Chapter 3
Methodology for the setpoint curve calculation

The SC can be generated manually by means of a hydraulic model. One of the most common computer programs for hydraulic analysis is EPANET [43]. Hence, the proposed methodology for computing SC will be explained in terms of this software. It is important to highlight that the goal of this method is to get the system head that met both pressure and demand requirements of the network. For that to be done, there is no need of defining any pumping station (i.e. the number of pumps, pump performance curves, efficiency curves). Moreover, it has to be considered that a single SC per pumping station is obtained as long as networks have no tanks. Otherwise, an envelope of points of different SCs is got.

Before the calculation, the following premises are assumed:

a) the pumping stations behave as nodes,
b) each supply source has an associated pumping station, and
c) not all pumping stations have related sources of supply (i.e. booster pumping stations).

If there are booster pumping stations in the network, some additional considerations must be followed as it will be shown in the cases of study. The method to calculate the SC will be different depending on pressure-driven demands (PDD), non-pressure driven demands (NPDD), flow rate limitations, the number of pumping stations as well as the storage capacity of the network. In this context, for both pressure dependent and no dependent demands, there are three general cases:

a) SC for a network with just one pumping station and without storage capacity.

b) SC for a network with more than one pumping station and without storage capacity.
c) SC for a network with one or more pumping stations and with storage capacity.

The second and third cases are part of the contributions of this research work. However, the third case will be presented in the section of optimisation of SC in networks with storage capacity.

3.1. Setpoint curve for a network with just one pumping station, without storage capacity, and non-pressure-driven demands

The first requirement is preparing a calibrated hydraulic model of the network. This will have only one not defined pumping station (i.e. number and size of pumps). There are no tanks. Therefore the analysis is performed in static state. It means that the whole procedure will be repeated as many times as the network demand changes, i.e. for the total number of demand stages. Usually, any change in demand is associated with a period, so for descriptive purposes whenever reference is made to the demand of the network it will be indicated as the period of analysis *i*. For each period *i* one point of the SC will be calculated. The steps to determine the SC are those collected schematically in Figure 23.

Since pumping station is not known yet, this will be represented as a node. In fact, the node will work as the discharge node of the pumping station. The idea is to determine the hydraulic grade line (HGL) elevations at discharge node for each demand of the network. Although at the end, the HGL at suction node has to be taken away from the HGL elevation at the discharge node to compute the pressure head or setpoint head of the pumping station. There are several types of nodes in EPANET (i.e. consumption nodes, tanks, reservoirs), for the purposes of the method it will be a reservoir. For sure, this last assumption does not have physical sense, but it will be used for mathematical purposes.

It is worth to mention that as there is just one pumping station, the flow to be supplied by the pumping station (Q_i) will be the same as the total flow demand (TFD_i) of the network in each period. Each one of the steps to complete the SC can be enumerated as follows:

1. Set the network demand for the period of simulation *i.*
2. Before solving the hydraulics for a specific value of the total flow demand (that is, the same flow rate supplied by the pumping station), an initial arbitrary elevation must be assigned to the reservoir ($Hd_{i,0}$). The goal is obtaining information of the network. Hence, the initial value of the elevation is not important. As a recommendation it can be higher than the minimum pressure required.

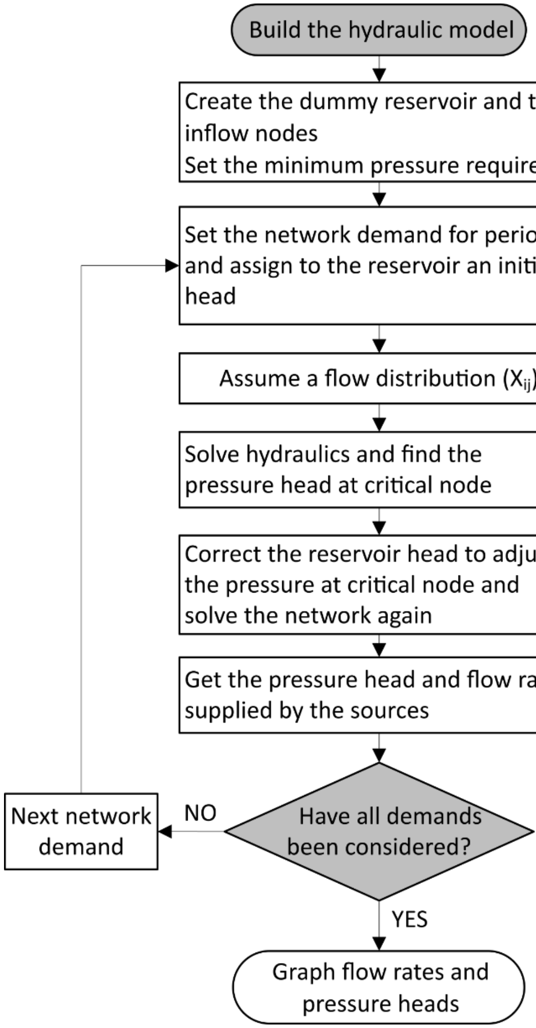

Figure 23. Flowchart for setpoint calculation of one pumping station

3. Then, the pressure heads of each node will be found as well as the pressure head at the critical node (PHc_i).
4. The next step is to check whether the condition of minimum pressure head required (PH_{min}) on the network is accomplished or not. Thus, the pressure head difference between both values is found.

$$\Delta PH_i = PH_{min} - PHc_i \tag{39}$$

5. One of the following statements must be accomplished: a) If $PHc_i > PH_{min}$ the elevation of the reservoir has to be reduced and b) If $PHc_i < PH_{min}$ the elevation of

reservoir has to be increased. The head is adjusted until both PHc_i and PH_{min} values are the same.

$$Hd_i = Hd_{i,0} + \Delta PH_i \quad (40)$$

6. It is important to remember that, up to now only HGL elevation at discharge node (Hd_i) has been got. Thus, the HGL elevation at suction node of pumping station (Hs_i) must be taken away from Hd_i for computing the setpoint head (PH_i) for the demand (Q_i) under analysis.

$$PH_i = Hd_i - Hs_i \quad (41)$$

7. Both values Q_i and PH_i are recorded.
8. Set the next demand *i* on the network until reach the total number of demand stages (*Nst*). The SC will have as many points as values of Q_i will be analysed.
9. Finally, all values of Q_i and PH_i got for the total number of stages are drawn.

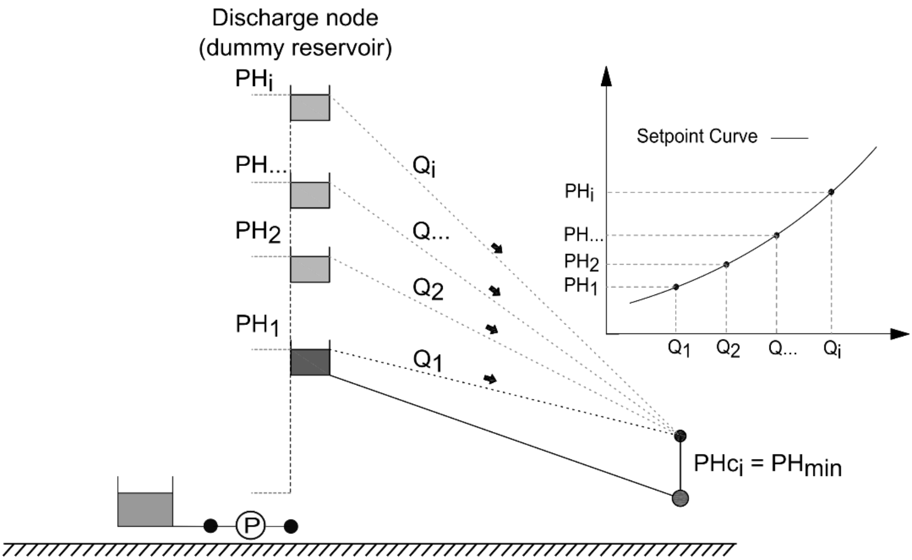

Figure 24. Scheme of the process to construct the setpoint curve for a WDN.

A schematic representation of the whole process of the SC calculation can be seen in Figure 24. The system is formed by one pumping station represented as a reservoir and one demand node. The figure shows the function of the dummy reservoir to compute the setpoint heads. In this case, the variation of the reservoir head allows adjusting the pressure in the demand node. Besides, reservoir head represents the HGL elevation at the discharge node of the pump. Finally, it can be observed that the minimum pressure head is kept at the critical node as a constant value each time the demand changes.

Example 1. Academic network A1

The technique previously introduced could be understood in a better way using an academic distribution network modelled in EPANET. There is just one pumping station (PS1) that has been represented as a dummy reservoir (Figure 25). There are no tanks. Head losses will be calculated by using Hazen-Williams. All pipelines have a coefficient C_{HW} = 140. The daily average flow rate demand is 100 l/s. The minimum pressure required in the network is 20 m, and water consumption does not depend on the pressure.

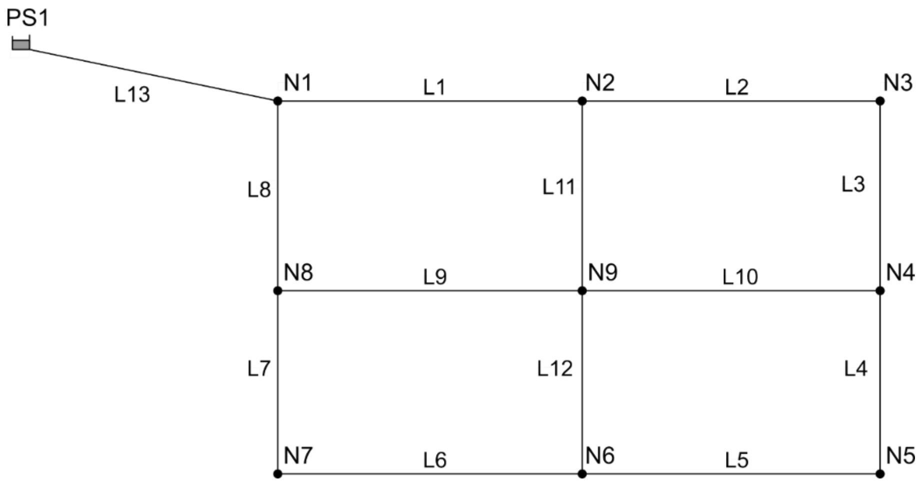

Figure 25. A1 network with pumping station PS1

The system has 9 junctions, 13 pipelines. The junctions have an average elevation of 8 m (Table 5). It is assumed that the HGL elevation at suction for PS1 is zero.

Table 5. Example network 1. Junctions.

Node ID	Elevation	Base Demand	Node ID	Elevation	Base Demand
	m	LPS		m	LPS
Junc N1	10	10	**Junc N6**	4	20
Junc N2	15	10	**Junc N7**	12	10
Junc N3	7	10	**Junc N8**	14	10
Junc N4	5	10	**Junc N9**	5	10
Junc N5	4	10	**Reservoir PS1**	1	-

Almost all the pipes have the same diameter. The only difference is in the pipeline L13 that is the injection line of the water source (Table 6).

Table 6. Example network 1. Links

Link ID	Length	Diameter	Roughness
	m	mm	
Pipe L1	1000	160	140
Pipe L2	1000	160	140
Pipe L3	1000	160	140
Pipe L4	1000	160	140
Pipe L5	1000	160	140
Pipe L6	1000	160	140
Pipe L7	1000	160	140
Pipe L8	1000	160	140
Pipe L9	1000	160	140
Pipe L10	1000	160	140
Pipe L11	1000	160	140
Pipe L12	1000	160	140
Pipe L13	1000	300	140

The demand pattern for a period of 24 h is presented by Figure 26.

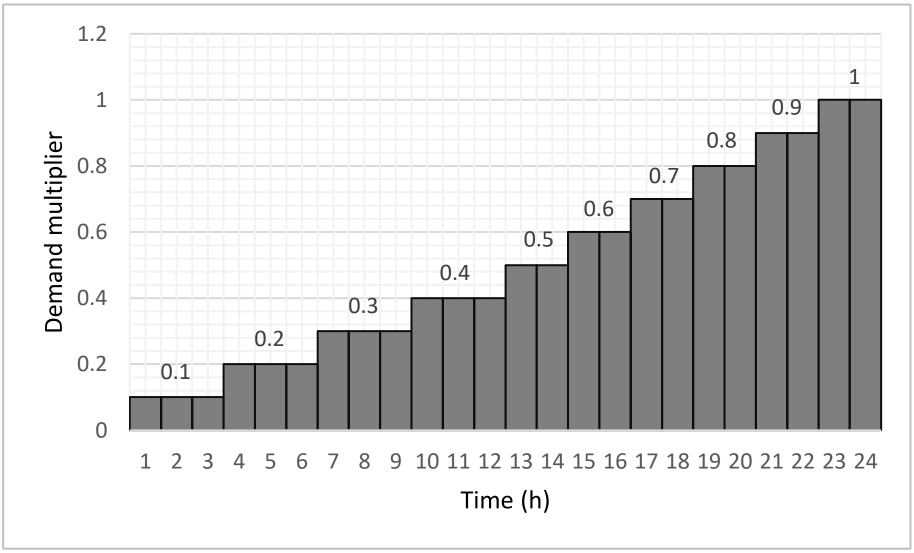

Figure 26. Demand pattern

All the calculations made are presented in Table 7. In column 1 is indicated the period of simulation and in column 2 the total flow demand. Before solving the hydraulics, a starting arbitrary elevation has been allocated to the pumping station (PS1). This value is 45 m (column 3) and is the same for all the demand changes, although it could be any value. Then, the network must be solved. Next step consists of identifying critical nodes (column 4) as well as their pressure head values (column 5). At this point, it is required to check whether the minimum pressure condition (column 6) is accomplished. In column 7 the deficit or excess of pressure at the critical node is shown (Equation 39). Then, the elevation head (i.e. pumping station) is corrected using Equation (40) (column 8). Now, should a new analysis is performed, the pressure at the critical node will be the minimum allowed, in this case, 20 m.

Finally, the SC is got by representing graphically the total flow demand (column 2) versus the pressure head at the reservoir (column 8) as it can be seen in Figure 27. It is worth to remember that HGL elevation at suction node is zero, hence the HGL elevation at discharge node of the pumping station (i.e. reservoir PS1) is equal to the system head. In Figure 27, it can be observed that, despite the variation of the critical node (Table 7), only one SC is obtained.

Table 7. Setpoint curve of A1 network with non-pressure-driven demands

Time	**Total Flow Demand (TFD_i)**	**$Hd_{i,0}$ (PS1)**	**ID critical node**	**PHc_i**	**PH_{min}**	**ΔPH**	**PH_i (S1) correction**
(h)	(L/s)	(m)		(m)	(m)	(m)	(m)
(1)	(2)	(3)	(4)	(5)	(6)	(7) = (6) - (5)	(8) = (3) + (7)
1-3	10	45	N2	29.54	20	-9.54	35.46
4-6	20	45	N2	28.34	20	-8.34	36.66
7-9	30	45	N2	26.48	20	-6.48	38.52
10-12	40	45	N2	24.00	20	-4.00	41.00
13-14	50	45	N2	20.93	20	-0.93	44.07
15-16	60	45	N2	17.29	20	2.71	47.71
17-18	70	45	N2	13.09	20	6.91	51.91
19-20	80	45	N7	7.51	20	12.49	57.49
21-22	90	45	N7	1.30	20	18.70	63.70
23-24	100	45	N7	-5.53	20	25.53	70.53

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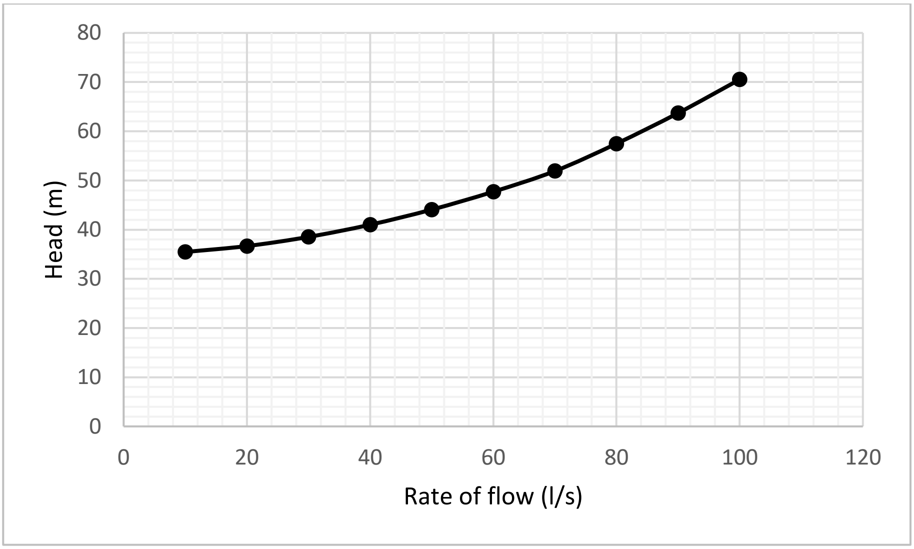

Figure 27. Setpoint curve A1 network

3.2. Setpoint curve for a network with just one pumping station, without storage capacity, and pressure-driven demands

In the case of PDD, emitter coefficients will be used at consumption nodes. Emitters are elements that relate flow and the pressure head existent upstream. The difficulty apparently lies in how to determine those coefficients and where to place them. If a node represents a sector, its pressure head will be given by the average pressure of it. The relation is provided by the following expression [47], [58]:

$$Q = C \cdot P^{\alpha} \quad \textbf{(42)}$$

Where,

Q leakage flow,
C emitter coefficient,
P average zone pressure,
∝ emitter exponent.

For nozzles and sprinkler heads, the emitter exponent has a value of 0.5 [43]. The methodology is quite like the case when consumption does not depend on pressure. The difference lies in that correction of the reservoir head has to be done more than once. When head of the reservoir is changed, the total flow demand also changes, and it has to be recalculated. Obviously, the demand changes as a result of the variation in pressure head at each node in the network. In this sense, pressure head at the critical node does not remain constant. Thus, reservoir head must be adjusted several times until pressure head at critical node meets the minimum pressure required. In this way, the process of correction of the elevation at reservoir becomes iterative (Figure 28).

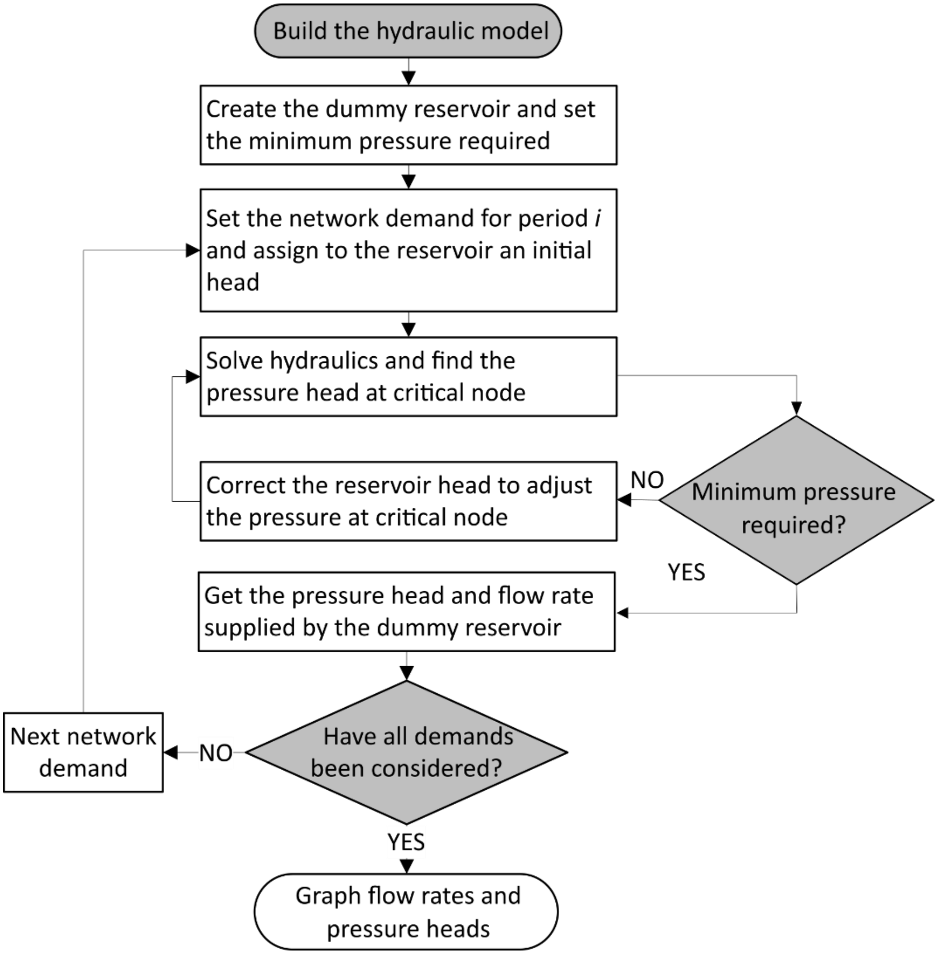

Figure 28. Setpoint curve calculation for a network with one pumping station and pressure-driven demands

To clarify any doubt about the steps to follow, they will be enumerated below:

1. Set the demand for the nodes for simulation period *i*.
2. The emitter coefficients at each water demand node must be specified.
3. An initial arbitrary head has to be assigned to the reservoir ($Hd_{i,0}$). This allows to obtain feasible solutions of the network to their later analysis.

4. Solve the hydraulics and find the critical node as well as its pressure head.
5. Determine whether there is deficit or excess of pressure at critical node (Equation 39). If pressure head at critical node is the same as the minimum pressure required, then go to step 7.
6. Correct the reservoir head (Hd_i) by using Equation (40).
7. Repeat step 2 matching the corrected value of the reservoir head as the initial head.

$$Hd_{i,0} \approx Hd_i \quad (43)$$

8. Compute the pressure head at pumping station (Equation 41).
9. Recalculate the total flow demand for the period *i* of analysis. There is just one pumping station, so the pump discharge Q_i is the same as TFD_i
10. Write down both values Q_i and PH_i
11. Repeat the analysis for the total number of scenarios (*Nst*), i.e. for all the network demands.
12. Finally, draw the SC.

Example 2. Academic network A1 with emitters

Following on from the instance 1 and considering pressure dependent consumption, two new coefficients will be assumed. The first one is the emitter coefficient for all nodes which value will be 0.8. The second one will be the emitter exponent, and its value will be 0.5.

The baseline information is the same as in the A1 network (Figure 25). It has been considered a minimum pressure required of 20 m. The results are shown in Table 8 as a demonstration of the calculation work.

Table 8. Setpoint curve of A1 network with pressure-driven-demands

Time (h)	Iterations number	PH_i (PS1) (m)	ID critical node	PHc_i (m)	Ph_{min} (m)	ΔPH_i (m)	PH_i (S1) correction (m)	TFD_i (L/s)
(1)	(2)	(3)	(4)	(5)	(6)	(7) = (6) - (5)	(8) = (3) + (7)	(9)
1-3	1	45	N2	21.77	20	-1.77	43.23	48.09
	2	43.23	N2	20.32	20	-0.32	42.91	47.09
	3	42.91	N2	20.06	20	-0.06	42.85	46.9
	4	42.85	N2	20.01	20	-0.01	42.84	46.87
	5	**42.84**	**N2**	**20**	**20**	**0.00**	**42.84**	**46.86**
4-6	1	45	N2	19.07	20	0.93	45.93	56.12
	2	45.93	N2	19.81	20	0.19	46.12	56.65

Time (h)	Iterations number	PH_i (PS1) (m)	ID critical node	PHc_i (m)	Ph_{min} (m)	ΔPH_i (m)	PH_i (S1) correction (m)	TFD_i (L/s)
(1)	(2)	(3)	(4)	(5)	(6)	(7) = (6) - (5)	(8) = (3) + (7)	(9)
	3	46.12	N2	19.96	20	0.04	46.16	56.75
	4	46.16	N2	19.99	20	0.01	46.17	56.78
	5	**46.17**	**N2**	**20**	**20**	**0.00**	**46.17**	**56.78**
7-9	1	45	N2	16.15	20	3.85	48.85	63.84
	2	48.85	N2	19.1	20	0.90	49.75	66.04
	3	49.75	N2	19.8	20	0.20	49.95	66.53
	4	49.95	N2	19.95	20	0.05	50.00	66.64
	5	50.00	N2	19.99	20	0.01	50.01	66.67
	6	**50.01**	**N2**	**20**	**20**	**0.00**	**50.01**	**66.68**
10-12	1	45	N2	13.07	20	6.93	51.93	71.24
	2	51.93	N7	18.12	20	1.88	53.81	76.24
	3	53.81	N7	19.45	20	0.55	54.36	76.26
	4	54.36	N7	19.84	20	0.16	54.52	76.55
	5	54.52	N7	19.95	20	0.05	54.57	76.63
	6	54.57	N7	19.99	20	0.01	54.58	76.66
	7	**54.58**	**N7**	**20**	**20**	**0.00**	**54.58**	**76.66**
13-14	1	45	N7	9.58	20	10.42	55.42	78.26
	2	55.42	N7	16.47	20	3.53	58.95	84.39
	3	58.95	N7	18.88	20	1.12	60.07	86.26
	4	60.07	N7	19.65	20	0.35	60.42	86.84
	5	60.42	N7	19.89	20	0.11	60.53	87.02
	6	60.53	N7	19.97	20	0.03	60.56	87.08
	7	60.56	N7	19.99	20	0.01	60.57	87.09
	8	60.57	N7	19.99	20	0.01	60.58	87.1
	9	**60.57**	**N7**	**20**	**20**	**0.00**	**60.57**	**87.1**
15-16	1	45	N7	5.88	20	14.12	59.12	84.84
	2	59.12	N7	14.65	20	5.35	64.47	93.39
	3	64.47	N7	18.16	20	1.84	66.31	96.19
	4	66.31	N7	19.39	20	0.61	66.92	97.11
	5	66.92	N7	19.79	20	0.21	67.13	97.42

Time (h)	**Iterations number**	**PH_i (PS1)** (m)	**ID critical node**	**PHc_i** (m)	**Ph_{min}** (m)	**ΔPH_i** (m)	**PH_i (S1) correction** (m)	**TFD_i** (L/s)
(1)	(2)	(3)	(4)	(5)	(6)	(7) = (6) - (5)	(8) = (3) + (7)	(9)
	6	67.13	N7	19.93	20	0.07	67.20	97.52
	7	67.2	N7	19.98	20	0.02	67.22	97.55
	8	67.22	N7	19.99	20	0.01	67.23	97.56
	9	**67.23**	**N7**	**20**	**20**	**0.00**	**67.23**	**97.57**
17-18	1	45	N7	2.35	20	17.65	62.65	90.85
	2	62.65	N7	12.46	20	7.54	70.19	102.03
	3	70.19	N7	17.21	20	2.79	72.98	105.95
	4	72.98	N7	19	20	1.00	73.98	107.32
	5	73.98	N7	19.65	20	0.35	74.33	107.8
	6	74.33	N7	19.88	20	0.12	74.45	107.96
	7	74.45	N7	19.96	20	0.04	74.49	108.02
	8	74.49	N7	19.98	20	0.02	74.51	108.04
	9	**74.51**	**N7**	**20**	**20**	**0.00**	**74.51**	**108.05**
19-20	1	45	N7	-0.36	20	20.36	65.36	95.7
	2	65.36	N7	9.59	20	10.41	75.77	109.96
	3	75.77	N7	15.83	20	4.17	79.94	115.39
	4	79.94	N7	18.42	20	1.58	81.52	117.39
	5	81.52	N7	19.42	20	0.58	82.10	118.13
	6	82.1	N7	19.79	20	0.21	82.31	118.4
	7	82.31	N7	19.92	20	0.08	82.39	118.5
	8	82.39	N7	19.97	20	0.03	82.42	118.53
	9	82.42	N7	19.99	20	0.01	82.43	118.55
	10	82.43	N7	19.99	20	0.01	82.44	118.55
	11	**82.44**	**N7**	**20.00**	**20**	**0.00**	**82.44**	**118.56**
21-22	1	45	N7	-2.87	20	22.87	67.87	99.72
	2	67.87	N7	6.54	20	13.46	81.33	117.46
	3	81.33	N7	14.16	20	5.84	87.17	124.58
	4	87.17	N7	17.66	20	2.34	89.51	127.35
	5	89.51	N7	19.09	20	0.91	90.42	128.41
	6	90.42	N7	19.65	20	0.35	90.77	128.82

Time **(h)**	**Iterations number**	**PH_i (PS1)** **(m)**	**ID critical node**	**PHc_i** **(m)**	**Ph_{min}** **(m)**	**ΔPH_i** **(m)**	**PH_i (S1) correction** **(m)**	**TFD_i** **(L/s)**
(1)	(2)	(3)	(4)	(5)	(6)	(7) = (6) - (5)	(8) = (3) + (7)	(9)
	7	90.77	N7	19.87	20	0.13	90.90	128.98
	8	90.9	N7	19.95	20	0.05	90.95	129.03
	9	90.95	N7	19.98	20	0.02	90.97	129.06
	10	90.97	N7	19.99	20	0.01	90.98	129.07
	11	**90.98**	**N7**	**20.00**	**20**	**0.00**	**90.98**	**129.07**
23-24	1	45	N7	-5.28	20	25.28	70.28	103.54
	2	70.28	N7	3.48	20	16.52	86.80	124.54
	3	86.6	N7	12.08	20	7.92	94.52	133.4
	4	94.52	N7	16.65	20	3.35	97.87	137.12
	5	97.87	N7	18.63	20	1.37	99.24	138.61
	6	99.24	N7	19.45	20	0.55	99.79	139.21
	7	99.79	N7	19.78	20	0.22	100.01	139.45
	8	100.01	N7	19.92	20	0.08	100.09	139.54
	9	100.09	N7	19.96	20	0.04	100.13	139.58
	10	100.13	N7	19.99	20	0.01	100.14	139.6
	11	100.14	N7	19.99	20	0.01	100.15	139.6
	12	**100.15**	**N7**	**20.00**	**20**	**0.00**	**100.15**	**139.6**

The SC is got by drawing the flow rate (column 9) and pressure head (8) at the end of the iteration process for each period of analysis (Figure 29). The summary of the SCs points is presented in Table 9.

Table 9. A1 network, setpoint curve points

Time (h)	Ph_i (S1) (m)	Q_i (S1) (l/s)
1-3	46.86	42.84
4-6	56.78	46.17
7-9	66.68	50.01
10-12	76.66	54.58
13-14	87.10	60.57
15-16	97.57	67.23
17-18	108.05	74.51
19-20	118.56	82.44
21-22	129.07	90.98
23-24	139.60	100.15

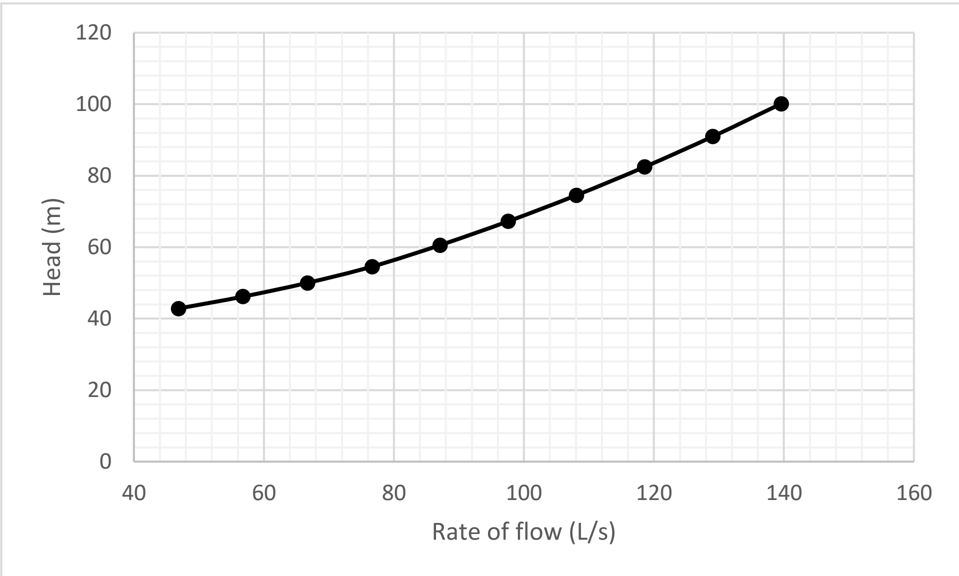

Figure 29. A1 network. Setpoint curve for pressure driven demands

3.3. Setpoint curve for a network with more than one pumping station, without storage capacity, and non-pressure-driven demands

This section consists of a generalisation of the case above. Therefore, before getting the SCs for the pumping stations of a network, it is essential to understand fully the first instance where there is just one pumping station. The analysis will be carried out in static state, and a point of the SC will be got for each point *i* of the demand pattern of the network. Besides, it is assumed that pumping stations are linked to water sources that will supply water to the system.

As in the previous case, there is no need of define the pumping stations (i.e. size, number, pump performance curves, etc.). Thus, all pumping stations will be represented as nodes. There is a number *Nps* of pumping stations with *j* elements. Hence, the aim of the method is to find the pressure head (PH_{ij}) at pumping station *j* for each discharge (Q_{ij}) at time *i*. Thus, there will be one SC for each pumping station *j*.

As in the previous case, one of the nodes (i.e. pumping stations) will be a reservoir. There has to be at least one reservoir in the network as a condition to solve the hydraulics. For purposes of the methodology, there will be just one (i.e. the dummy reservoir). Anyone of the *Nps* pumping stations can be selected to be the reservoir. However, the remaining nodes (*Nps-1*) will be inflow nodes (i.e. consumption nodes with negative demand). Hence, in addition to the head of the reservoir, the inflow at remaining pumping stations are also variables to be set. A diagram of the methodology explained below is shown in Figure 30.

Before solving hydraulics, some previous steps must be followed:

1. First of all, the base demand on networks nodes for the period *i* must be established.
2. An initial arbitrary elevation must be assigned to the reservoir. Only in the case of the reservoir, the elevation corresponds to the HGL elevation at discharge node. Therefore, it has to be remembered to subtract the HGL elevation at the suction node at the end, before getting the final setpoint head of the reservoir. The initial value of the elevations serves just to obtain an initial solution of the network.
3. In the case of the injection nodes, the inflow must be allocated $\left(Q_{ij}\right)$. The level height of nodes will be given by the HGL elevation at suction node of each pumping station *j-1*, that are starting data. The discharge of pumping stations *Nps-1* can be computed as a percentage X_{ij} of the total flow demand at time *i*. The values of X_{ij} can remain fixed over the whole simulation period or, they can be variable depending on working conditions imposed in each pumping station.

$$Q_{ij} = -X_{ij} \cdot TFD_i \tag{44}$$

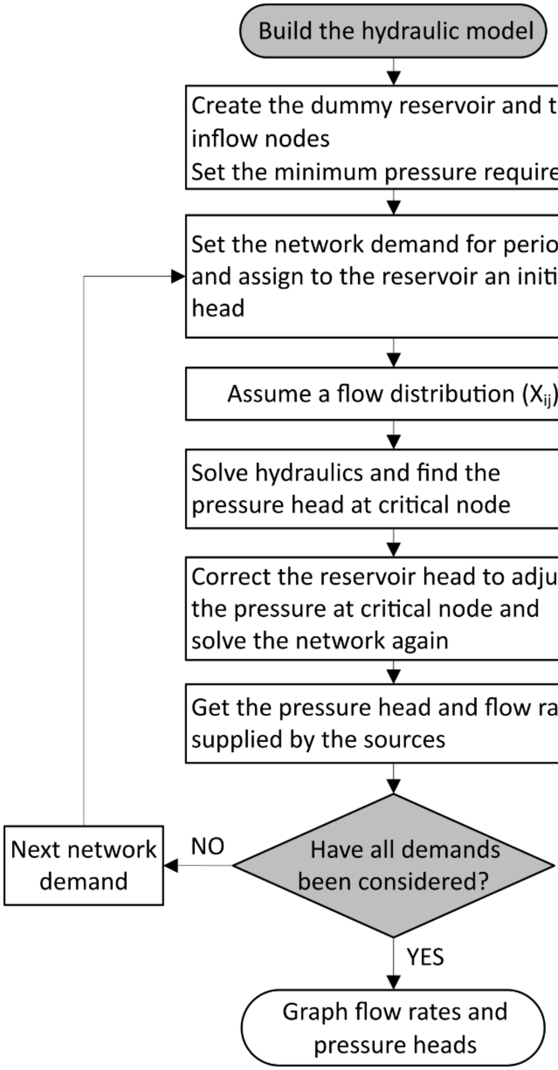

Figure 30. Flowchart for setpoint calculation process in a network in more than one pumping station and non-pressure-driven demands

When there are flow restrictions: minimum flow ($Qmin_{ij}$) or maximum flow ($Qmax_{ij}$), Equation (45) is applied. It should be notice that this kind of restrictions only can be applied to inflow nodes.

$$Q_{ij} = -\left[Qmin_{ij} + X_{ij} \cdot \left(Qmax_{ij} - Qmin_{ij}\right)\right] \quad \textbf{(45)}$$

The sum of flow rates to be supplied by each pumping station for the period of analysis must be equal to the total flow demand for the same time. In that sense, either Equation (44) or (45) must accomplish the condition of Equation (46).

$$\sum_{j=1}^{Nps} Q_{ij} = TFD_i \tag{46}$$

Next, the hydraulics is solved and it is proceeded as in the case of one pumping station. This can be summarised as follows:

4. The pressure head at critical node must be defined and contrasted with the minimum pressure required. Thus, the difference between pressure heads must be found ΔPH_i as in Equation (39).
5. The elevation at reservoir has to be corrected according to the result of the previous step until the condition $PHc_i = PH_{min}$ is reached.
6. In the reservoir, the HGL elevation at suction node must be subtracted from HGL elevation at discharge node. The last one is the elevation of the reservoir that is corrected when is needed.
7. The points (i.e. values of PH_{ij} and Q_{ij}) of the SC for the TFD_i and that belongs to each one of pumping stations are recorded.
8. The analysis will be performed for each value *i* of the total flow demand to get all the points of the SCs.

In the case of a network with booster pumping stations, the alternative is to split the system in the places where they are located. In this way, there will be two nodes instead of one for representing the booster stations (Figure 31). The first node will be the suction node of the pumping station (point A), and the second one will be the discharge node (point B). Therefore, there will be two separate networks or more depending on the case.

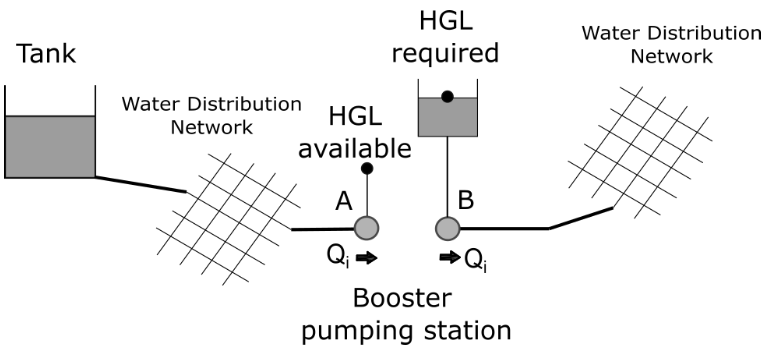

Figure 31. Booster pumping station representation

To connect the two parts of the network, the point A will be assumed as a demand node and its demand will be the same as the flow supplied by node B. However, the node B will be assumed as a reservoir (i.e. head node). This means, that the separate network will have as the source of supply the node B. Therefore, the part of the network with the booster-pumping station can be solved exactly as in the case of a SC for a network with just one pumping station and without tanks. After solving the whole system, HGL elevations at both suction and discharge node will be defined. The pressure head at booster pumping station will be given by the difference between both values. In the case of the HGL available be bigger than the HGL required, the booster pumping is not needed.

The flow allocation among several pumping stations will depend on the requirements of the network and have to be established at the beginning of the analysis. It is worth to mention that the distribution of flow rates is only possible for pumping stations linked with water sources. Therefore, when there are both types, pumping stations associated with water sources and as booster stations, it has to be taken into account not to inject more water than the network really needs.

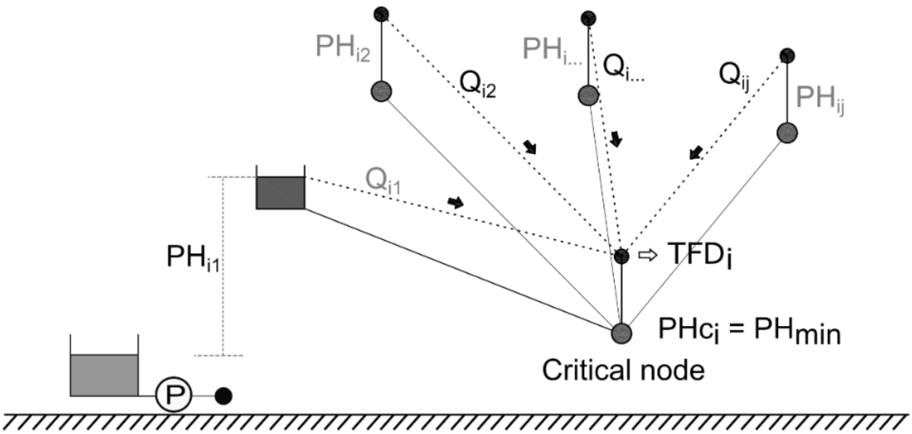

Figure 32. Setpoint curve computing process for networks with more than one pumping station

A scheme of the SC calculation process is shown in Figure 32. Figure shows a network with four pumping stations and one demand node. One of the pumping stations is represented by a dummy reservoir and the others are inflow nodes. It has to be noted that in the case of the reservoir the head is allocated, but for remaining nodes, the flows are assigned. In this way, there will be two types of output information. In the case of the reservoir (i.e. pumping station) the flow rate that is supplied to the network. And in the case of the inflow nodes the pressure head that is required to provide the flow rate needed under the preestablished conditions. Besides, independently of the flow assigned to the inflow nodes the pressure head of each one of them is adjusted by the dummy reservoir taking as a reference the pressure head at critical node. Thus, the pressure head at critical node is kept as constant for every scenario of analysis.

It is worth to mention that the obtained pressure heads at inflow nodes and at the dummy reservoir are the direct consequence of the flow distribution assigned to the inflow nodes. Thus, this combination of values (flow distribution and pressure heads) is unique and does not depend on the location of the dummy reservoir. This is, whatever the location of the dummy reservoir is, the result will be the same. This while the flow distribution and minimum pressure required do no change. In that sense, it can be said that the dummy reservoir serves only for mathematical purposes.

Example 3: TF network without emitters

To implement the previously introduced methodology, the TF network will be used (Figure 33).

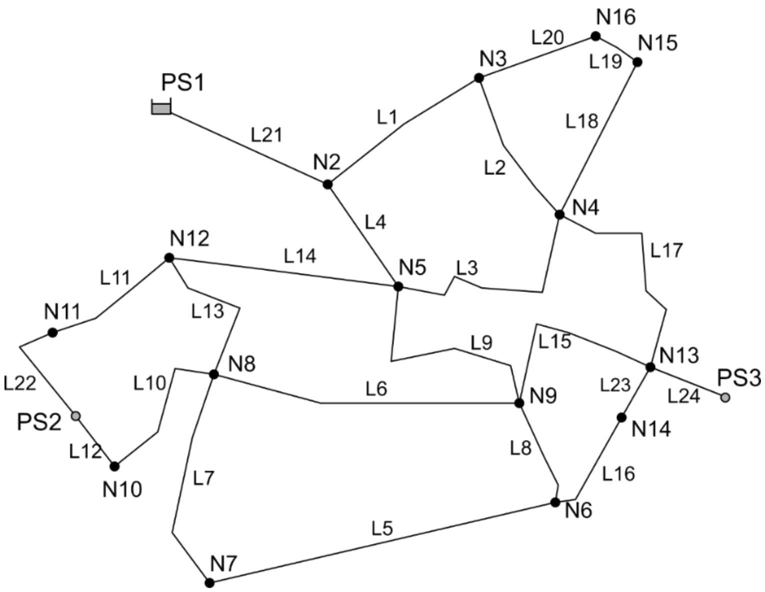

Figure 33. TF network

This system has three water sources linked with pumping stations. All of them will be defined as follows:

- Water source 1: pumping station PS1 represented as a dummy reservoir.
- Water source 2: pumping station PS2 represented as inflow node.
- Water source 3: pumping station PS3 represented as inflow node.

The minimum pressure allowed is PH_{min} = 20 m. The head losses will be computed by mean of Hazen-Williams equation. It will be assumed a coefficient C_{HW} = 140 for all pipelines. The junction elevations and base demands are presented in Table 10. The HGL elevation at suction nodes for pumping stations will be: PS1 = 0 m, PS2 = 4 m and PS3 = 0 m. The average daily flow demand is 100 l/s. The length and diameter of the pipelines are shown in Table 11.

Table 10. TF network. Junctions information

Node ID	Elevat. (m)	Base Demand (LPS)	Node ID	Elevat. (m)	Base Demand (LPS)	Node ID	Elevat. (m)	Base Demand (LPS)
N2	8	5	**N8**	5	7	**N14**	4	2
N3	8	4	**N9**	6	10	**N15**	3	10
N4	5	3	**N10**	2	9	**N16**	3	15
N5	8	4	**N11**	7	5	**PS2**	4	0
N6	4	3	**N12**	7	10	**PS3**	0	0
N7	2	8	**N13**	5	5	**PS1**	45	-

Table 11. TF network. Pipelines information

Link ID	Leng. (m)	Diam. (mm)	Roug.	Link ID	Leng. (m)	Diam. (mm)	Roug.	Link ID	Leng. (m)	Diam. (mm)	Roug.
L1	200	150	140	**L9**	250	150	140	**L17**	98	60	140
L2	150	100	140	**L10**	300	100	140	**L18**	300	80	140
L3	150	100	140	**L11**	300	100	140	**L19**	500	80	140
L4	200	200	140	**L12**	125	100	140	**L20**	400	100	140
L5	200	60	140	**L13**	300	80	140	**L21**	1500	250	140
L6	400	80	140	**L14**	250	150	140	**L22**	125	100	140
L7	300	60	140	**L15**	250	80	140	**L23**	52	60	140
L8	300	80	140	**L16**	100	60	140	**L24**	1000	300	140

The demand pattern will be given by Figure 34. It has a duration of 24 h with time steps of one hour. The maximum demand occurs at midday between 12h00 and 15h00. And, the lowest demand happens between 0H00 and 3H00.

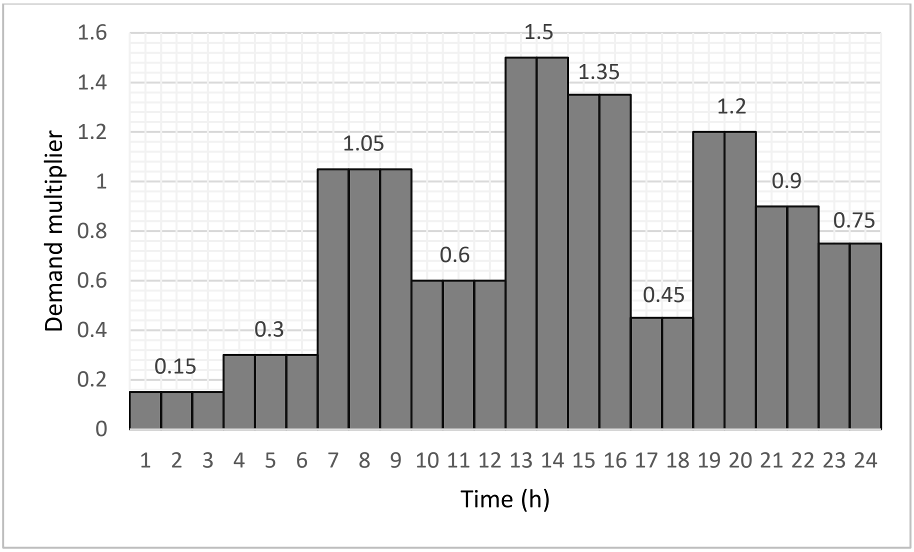

Figure 34. TF network. Demand pattern

All the initial information required for the SCs calculation is presented in Table 12. The first step is identifying the different periods of simulation (column 1) as well as the total flow demand for each one of them (column 2). Following the description of the process previously explained, an initial arbitrary elevation is assigned to the pumping station PS1. In this case will be 45 m (column 3). Then, flow rates must be assigned to the pumping stations, i.e. the inflows for nodes PS2 and PS3. For doing that, a distribution of discharges among pumping stations has to be assumed. For the present example, the next flow rate distribution will be considered:

$$Q_{i,PS2} = 30\% \cdot TFD_i \quad \textbf{(47)}$$

$$Q_{i,PS3} = 40\% \cdot TFD_i \quad \textbf{(48)}$$

The inflow for pumping station PS1 will be calculated as the remaining flow rate that is not supplied by the other pumping stations (column 6). Since pumping station PS1 run as a reservoir in the hydraulic model, it always will provide the deficit of flow which is not supplied by the other pumping stations. This is, PS1 does not have a flow limit. The flow to be provided by each pumping station (PS2 and PS3) are shown in columns 4 and 5 respectively.

Table 12. Starting information for setpoint curves calculation of Example 2 network

Time (h)	TFD_i (l/s)	$Hd_{i,0}$ (PS1) (m)	Q_i (PS2) (l/s)	Q_i (PS3) (l/s)	Q_i (PS1) (l/s)
(1)	(2)	(3)	(4) = 30%*(2)	(5) = 40%*(2)	(6) = (2) - (4) - (5)
1-3	15	45	4.5	6	4.5
4-6	30	45	9.0	12	9.0
7-9	105	45	31.5	42	31.5
10-12	60	45	18.0	24	18.0
13-14	150	45	45.0	60	45.0
15-16	135	45	40.5	54	40.5
17-18	45	45	13.5	18	13.5
19-20	120	45	36.0	48	36.0
21-22	90	45	27.0	36	27.0
23-24	75	45	22.5	30	22.5

Once all this information Table 12 has been set in the network, the hydraulics are solved. The resulting information is presented in Table 13. In column 7 critical nodes have been identified. It can be observed that critical nodes change over the periods of simulation according to the operating conditions of the network. There are even times when there are two critical nodes, both with the same pressure head. In this case, both have the same elevation. However, if they had different elevations the methodology is the same. This is, the method does not depend on the nodal elevation but on the lowest pressure head of the network. In column 8 pressure heads at critical nodes have been recorded. Then, it is required to check whether the minimum pressure condition is accomplished. In column 10 is shown either the excess or a deficit of pressure head at the critical node. The correction of the elevation of the reservoir (i.e. pumping station PS1) is made in column 11. Later, a new analysis of the network is performed to check that minimum pressure allowed at the critical node is kept. Finally, the pressure heads at pumping stations PS2 and PS3 that results of the model are write down (column 12 and 13).

The SC is obtained drawing the input and output information of flow rates and pressure heads. In the case of pumping station PS1 (column 6 vs column 11, Figure 35), for pumping station PS2 (column 12 vs column 4, Figure 36) and pumping station PS3 (column 13 vs column 5, Figure 37). Despite the demand curve has 24 time-steps, some of them are equal. Thus, the SCs only have 10 points. This is, the network has 10 different demand values.

Table 13. Calculation process for the setpoint curve of TF network

ID critical node	Phc_i (m)	Ph_{min} (m)	ΔPH_i (m)	PH_i (PS1) correction (m)	PH_i (PS2) (m)	PH_i (PS3) (m)
(7)	(8)	(9)	(10) = (9) - (8)	(11) = (3) + (10)	(12)	(13)
N3	36.90	20	-16.90	28.10	24.25	28.75
N3	36.65	20	-16.65	28.35	24.90	30.70
N15	22.82	20	-2.82	42.18	43.81	66.07
N15, N16	35.20	20	-15.20	29.80	27.80	38.28
N15, N16	4.88	20	15.12	60.12	67.02	106.37
N15, N16	11.46	20	8.54	53.54	58.50	91.59
N3	36.25	20	-16.25	28.75	25.92	33.72
N15	17.44	20	2.56	47.56	50.77	78.16
N15, N16	27.59	20	-7.59	37.41	37.64	55.37
N15, N16	31.72	20	-11.72	33.28	32.30	46.09

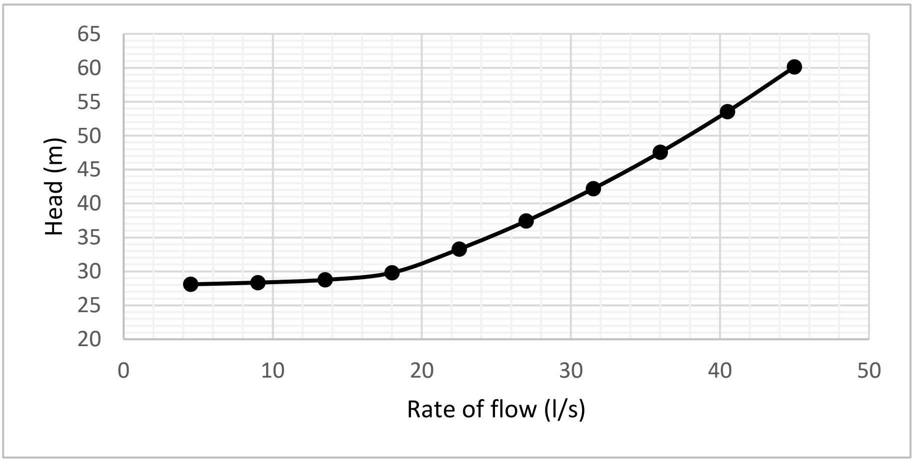

Figure 35. Setpoint curve PS1

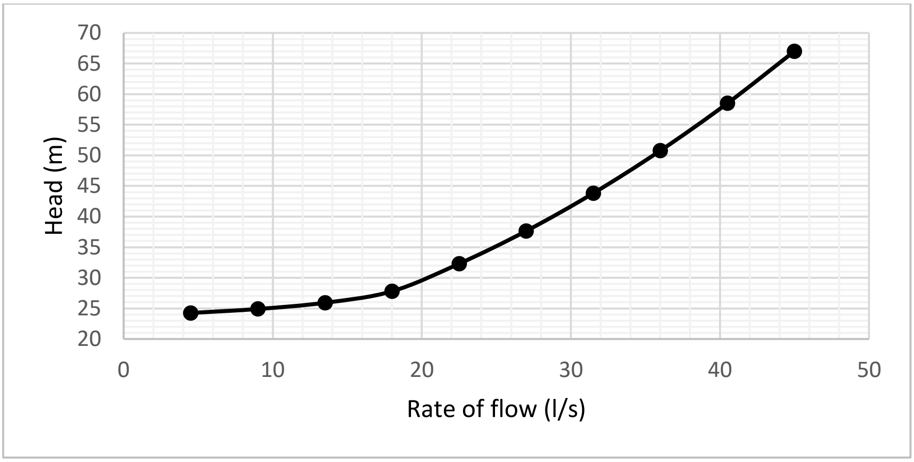

Figure 36. Setpoint curve PS2

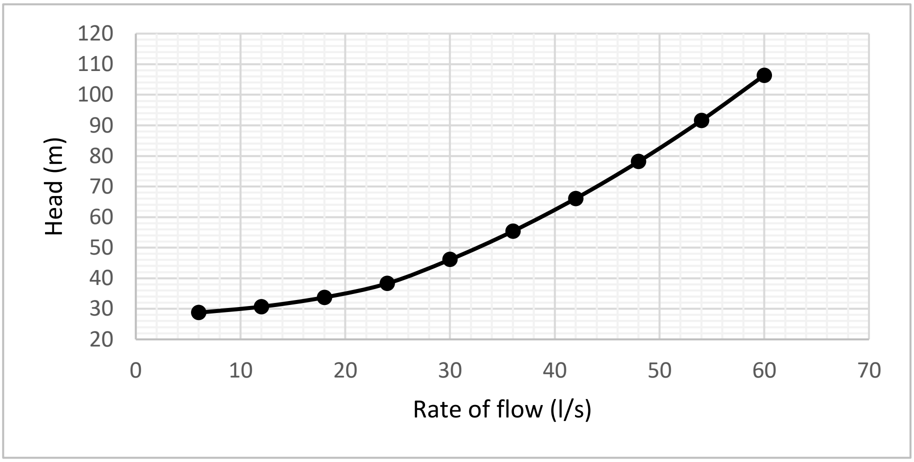

Figure 37. Setpoint curve PS3

It can be seen in the figures that the first three points of SCs have a different tendency to the other points. This because of the variation of the critical node. However, the method converges to only one SC, independently of the critical nodes variation.

3.4. Setpoint curve for a network with more than one pumping station, without storage capacity, and pressure-driven demands

In the example of SC computing for one pumping station and PDD, an iterative process for adjusting the minimum pressure at critical node of the network was carried out. This was made because of the variation of the total flow demand for each change of the elevation at dummy reservoir. In this case, besides the correction of the reservoir elevation to adjust the minimum pressure at the critical node, particular attention has to be given to both the total flow demand for each period and the flow rate distribution of the pumping stations. In that sense, the process to compute the SC can be separated in two parts. The first part is the *"pressure head and flow computation"* that corresponds to the previous and posterior steps before the hydraulic simulation. And the second part consists in the steps of *"correction of pressure and flow rate supplied"* before getting the final results.

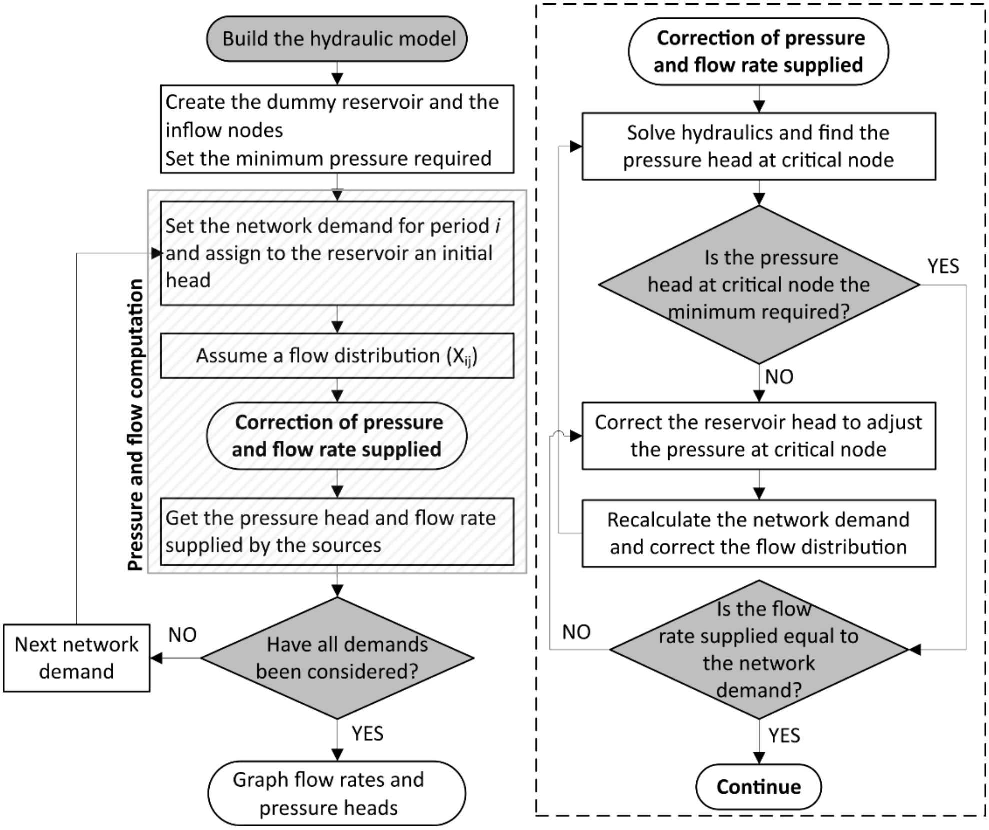

Figure 38. Diagram for setpoint curve calculation in the case of a water network with more than one pumping station and pressure dependent consumptions

A general diagram of the methodology to be described is shown in Figure 38. The next steps must be followed:

1. Set the nodes demand for the simulation period *i*.
2. The emitter coefficients are set at demand nodes.

3. Then, the total flow demand for time *i* has to be determined to fix later the supply distribution among the pumping stations (i.e. step 5). For the first iteration, the total flow demand value is unknown because of the PDD. Besides, the model has not been solved yet. Thus, it is assumed that the first value of TFD_i is given by the addition of the base demand at each node of the network. However, once that network has been solved the TFD_i must be recalculated. The recalculated value of TFD_i will be used as the starting value of the total flow demand in next iteration. In the end, at a specific moment, the TFD_i assumed at the beginning of the iteration will be equal to the TFD_i recalculated at the end of the same iteration.
4. Next, the initial head of the reservoir is assigned for time *i*. Its value is assigned arbitrary in order to get an initial solution of the network.
5. The flow distribution for *Nps-1* pumping stations is fixed (Equation 44 or 45).
6. The hydraulic model is solved.
7. The critical node has to be found as well as its pressure head.
8. The pressure head difference between the minimum pressure required and pressure head at critical node must be calculated.
9. The elevation at reservoir has to be corrected.
10. Determine the flow rate supplied by the dummy reservoir.
11. Recalculate the total flow demand by adding the supplied flows by the pumping stations.
12. Check that TFD_i assumed at the beginning of the iteration is equal to the recalculated TFD_i. The recalculated value will be the TFD_i assumed for next iteration.
13. At this point, two conditions must be accomplished. The first one is that difference between minimum pressure required, and pressure head at critical node must be zero ($PH_{min} = PHc_i$) and that the condition of step 12 is affirmative. Whether one of them is false, then go to step 14, but if both are true, then go directly to step 15.
14. Make $Hd_{i,0} \approx Hd_i$. Repeat from step 3.
15. Pressure heads and flow rates of all pumping stations are determined.
16. Set the demand for next *i* and repeat from step 3. Repeat the process until reach the total number of stages.
17. Finally, the SC for each pumping station is drawn.

Example 4: TF network with emitters

Following on from instance 3 (Figure 33) an emitter coefficient equal to 0.8 and an emitter exponent of 0.5 has been allocated to the consumption nodes. Emitters are not considered in non-demand nodes since leakage are not modelled. In any case, nodes that do not have emitters are those will be used as pumping stations.

For the simulation period *i* the iteration process always begins with the use of an arbitrary elevation value of the dummy reservoir (i.e. pumping station PS1). The flow rate distribution among the pumping stations is the same as example 3 (see, section 3.3.), 30% for pumping station PS2, 40% for pumping station PS3 and the missing flow rate discharge will be assumed by pumping station PS1. The minimum pressure head required is 20 m.

Table 14 and Table 15 show the starting information and resulting information respectively for the SC calculation for period 1-3. To clarify any doubt there could be about the process, iteration one will be explained. The first value of column (3) is the total flow demand without PDD. This is not the real demand, but it will be defined at the end of the iterative process. Then in column (4) an initial arbitrary elevation for the dummy reservoir is assumed. Column 5 and column 6 are the inflows allocated to pumping stations PS2 and PS3 respectively. From now on, it is required to solve the hydraulics. The critical node and its pressure are determined (column 7 and 8). The pressure head differential between the pressure head at the critical node and minimum pressure required is calculated (column 10). Then, the elevation of the dummy reservoir is corrected (column 11). Model outputs are pressure head at both pumping station PS2 and PS3 (column 12 and 13) as well as the flow rate supplied by the dummy reservoir (column 14). Finally, the real total flow demand is computed, this means the TFD_i is recalculated (column 15). The values of column 11 and column 15 are the base values for the next iteration (i.e. TFD_i and $Hd_{i,0}$). For the first analysis period, 8 iterations were needed. It can be observed that at iteration eight both conditions minimum pressure and flow demand are accomplished (i.e. step 13 of the method).

Table 14. TF network. Input information for setpoint curves calculation.

Time (h)	Iterations	TFD_i (l/s)	$Hd_{i,0}$ (PS1) (m)	Q_i (PS2) (l/s)	Q_i (PS3) (l/s)
(1)	(2)	(3)	(4)	(5) = 30%*(3)	(6) = 40%*(3)
1-3	1	15.00	45.00	4.50	6.00
	2	76.08	42.07	22.82	30.43
	3	84.74	31.08	25.42	33.90
	4	75.83	29.12	22.75	30.33
	5	72.65	29.40	21.79	29.06
	6	72.50	29.63	21.75	29.00
	7	72.71	29.66	21.81	29.08
	8	72.78	29.66	21.83	29.11

Table 15. TF Network. Output information of setpoint curve calculation.

ID critical node	PHc_i (m)	PH_{min} (m)	ΔPH_i (m)	PH_i(PS1) correction (m)	PH_i (PS2) (m)	PH_i (PS3) (m)	Q_i (PS1) (l/s)	TFD_i (l/s)
(7)	(8)	(9)	(10) = (9) - (8)	(11) = (3) + (10)	(12)	(13)	(14)	(15)
N7	22.93	20.00	2.93	42.07	26.72	31.88	65.58	76.08
N7	30.99	20.00	10.99	31.08	39.49	49.31	31.48	84.74
N3	21.96	20.00	1.96	29.12	32.45	44.53	16.51	75.83
N3	19.72	20.00	0.28	29.40	28.78	39.15	19.56	72.65
N3	19.77	20.00	0.23	29.63	28.31	38.07	21.64	72.50
N3	19.96	20.00	0.04	29.66	28.47	38.19	21.96	72.71
N3	20.01	20.00	0.01	29.66	28.55	38.30	21.88	72.78
N3	20.00	20.00	0.01	29.65	28.56	38.32	21.84	72.78

Since the procedure to compute all the points of the SC is repetitive, a summary of SC points is presented in Table 16.

Table 16. TF network. Setpoint curves for pressure dependent consumptions.

Time (h)	Q_i (PS1) (m)	PH_i (PS1) (l/s)	PH_i (PS2) (m)	Q_i (PS2) (l/s)	PH_i (PS3) (m)	Q_i (PS3) (l/s)
1-3	21.84	29.66	21.83	28.56	29.11	38.32
4-6	26.55	30.81	26.55	30.94	35.40	43.70
7-9	32.12	34.62	32.11	36.45	42.81	53.31
10-12	37.79	39.14	37.79	42.97	50.39	64.73
13-14	43.57	44.37	43.54	50.49	58.06	77.89
15-16	49.39	50.27	49.38	59.01	65.84	92.84
17-18	55.26	56.86	55.26	68.50	73.68	109.52
19-20	61.19	64.09	61.18	78.92	81.57	127.88
21-22	67.14	71.99	67.13	90.30	89.51	147.96
23-24	73.12	80.53	73.11	102.61	97.48	169.70

The SCs of pumping stations PS1 (i.e. dummy reservoir, Figure 39), PS2 (Figure 40) and PS3 (Figure 41) are presented following:

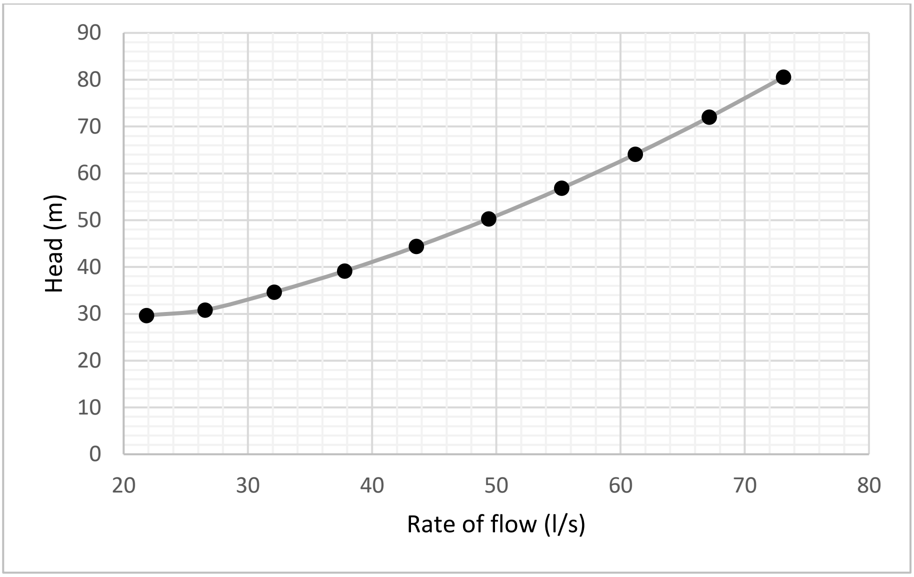

Figure 39. Setpoint curve PS1, pressure dependent consumptions

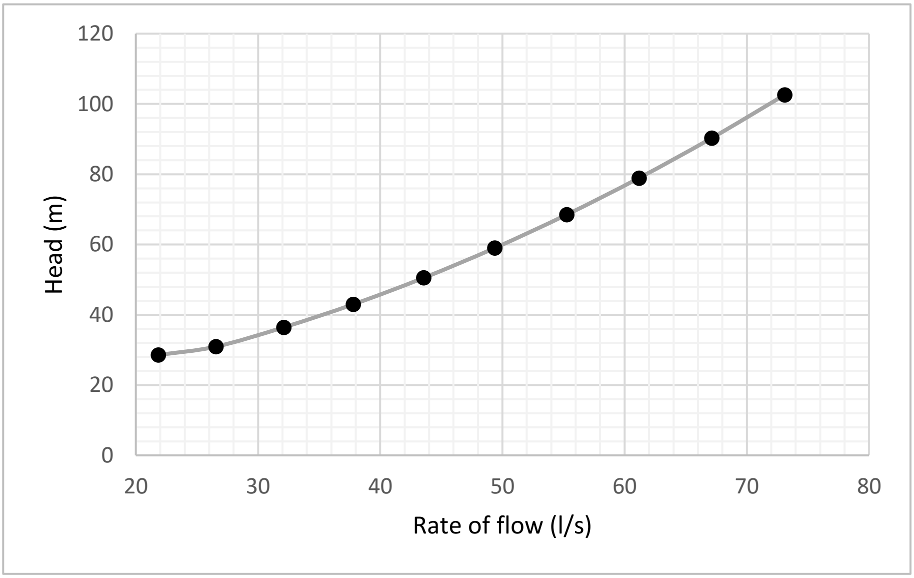

Figure 40. Setpoint curve PS2, pressure dependent consumptions

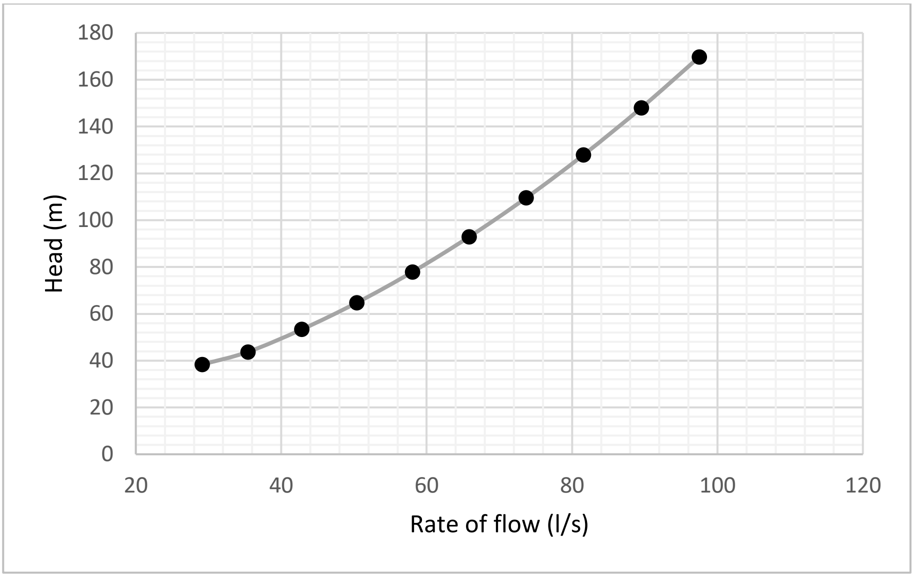

Figure 41. Setpoint curve PS3, pressure dependent consumptions

In Figure 39 can be observed the change of critical node from N3 to N7. This is reflected by the sudden change of gradient of the curve which happens after the first two points. Though the variation of the critical node affects all pumping stations, it is more difficult to locate such variation in the other figures (Figure 40 and Figure 41). Thus, it can be said that the variation of the critical node does not always introduce big fluctuations in the shape of the SC.

3.5. Problem associated with the flow distribution among pumping stations

In the case of the SC method calculation for more than one pumping station, some considerations have to be highlighted. They are the base of the present work of study and are fundamental to understand the development of the later sections.

It has to be noted that for several pumping stations the elevation of the dummy reservoir has to be fixed, but also the distribution of flow rates discharges among the pumping stations (Equation 44). This means that the discharge of pumps stations has to be predefined in order to compute the SC of each pumping station. Besides, the distribution stays uniform over the whole simulation and for each demand of the network. In that context, for each period of analysis, it is required a certain quantity of energy at pumping stations (i.e. pump head and flow rate) to meet both the network demand and minimum pressure head at the critical node. However, when there are changes in the flow distribution, the SCs will be different as well as the energy required. In that sense there are three possible situations:

a) There could be only one better constant distribution among pumping stations to minimise the pumping power required for the whole simulation period;
b) The flow distribution could be different for each stage of simulation; and
c) The flow distribution could stay constant for a flow rates range and change in the remaining flow rates.

Thus, the three statements above become an optimisation problem. This problem will be addressed in the Chapter 4 related with energy optimisation. The aim of that section is finding the optimal SCs that require the minimum energy on water networks with several pumping stations.

After finding the optimum flow distribution that produces the optimal SCs in energetic terms, it could be thought of moving into two different paths. The first one could be to propose a pumps system (i.e. sizing and pumps selection) to be adjusted as close as possible with the SCs trying to minimise costs both investment and operational. However, cost optimisation related directly to both sizing and pumps selections is out of the scope of this work. The other path consists in to minimise the costs by incorporating the energy costs to the SCs. This means that the aim of the study will evolve in to find the most economic SC independent of the pumps system that will be selected later. After all, the pump systems must be adapted to the most economical working conditions of the network and not the opposite. This second aim will be addressed in Chapter 5 where the cost optimisation is developed.

Finally, it can be noted that the methodology to compute the SC has been applied only in networks without storage capacity (i.e. tanks, deposits). That is because in a system with tanks is not possible to use a dummy reservoir to adjust the minimum pressure at the critical node. The reason is that instead of two constraints (i.e. pressure and demand), a third one has to be added, the storage constraint. Thus, tanks at the end of the simulation period should have the same or a higher storage level. In that sense, the pressure head at critical node could remain at the minimum expected over a specific time, but since the tanks are in higher points, the minimum pressure head on the network will increase to fill them. Consequently, it is not possible to know the pressure head of the critical node over the whole simulation period. Therefore, the approach will be different, but the concept of SC will be kept. Thus, the last part of the work will be about the energy and cost optimisation at pumping stations in networks with storage capacity by means of the SC concept (see, Chapter 6).

Chapter 4
Energy optimisation without storage capacity

As was mentioned before (see, section 2.4.), the setpoint curve (SC) represents the minimum energy required at pumping stations to meet the minimum pressure head needs of the network. In the case of several pump stations, it was also pointed out that the demand of the network must be distributed among pumping stations as a percentage of it. The flow distribution remains constant over the whole simulation period. However, it is known that different flow distributions mean different SCs. Thus, two aspects have to be kept in mind:

a) the SC marks the minimum energy consumption of the network (i.e. in terms of pressure), and
b) there are as many SCs as flow distributions.

Both aspects are fundamental to carry out the energy optimisation. In that sense, the problem consists in finding out which is the optimal flow distribution that leads to the optimal SC. The energy optimisation of the flows distribution will be done taking as a reference a specific objective function (OF) to be minimised. The terms to be included in the function are related directly to the method to compute the SC for water distribution networks (WDNs) with several pumping stations and without storage capacity.

It can be noted that assuming that the distribution is unknown, any of the pumping stations should be capable of supplying between 0 and 100% of the total demand in a specific period, as long as there are no external restrictions (i.e. maximum and minimum flow rate limit). The optimal flow distribution issue can be addressed, by mean of two approaches:

a) The discrete method (D-M), which tests every possibility of flows distribution within a limited set of alternatives.
b) The continuous method (C-M) where an optimisation algorithm is applied to find the optimal solution without the need of exploring every possible solution.

4.1. Problem formulation

The application of any of the two approaches requires constructing an OF. Hence, the OF involves the minimum energy consumption as a result of the sum of the product of the pressure heads and flow rates. To find the minimum value of the function, several flow rates combinations among the available pumping stations must be tested. The function is shown below:

$$min\, f(x)_{c,i} = \sum_{j=1}^{Nps-1} \left(X_{i,j,c} \cdot TFD_i \cdot PH_{i,j,c}\right) + \left(100 - \sum_{j=1}^{Nps-1} X_{i,j,c}\right) \cdot TFD_i \cdot PH_{i,resv,c} \tag{49}$$

$i = 1, \ldots, Nst$; $j = 1, \ldots, Nps\text{-}1$; $c = 1, \ldots, Nc$

Where,

$X_{i,j,c}$ is the percentage of the flow demand for the period *i*, and the pumping station *j* according to the combination *c;*
TFD_i is the total flow demand by the network for the period *i;*
$PH_{i,j,c}$ is the pressure head of the *Nps-1* pumping stations represented as inflows nodes over the period of simulation *i*, and the combination of distributions *c;*
$PH_{i,resv,c}$ is the pressure head of the dummy reservoir over the period simulation *i* and the combination of distributions *c;*
Nps is the total number of pumping stations on the network;
Nst is the total number of stages of analysis or periods of simulation;
Nc is the total number of combinations to be analysed.

The inflow flow rate of each pumping station will be defined by: Equation (50) in the case of the nodes, and Equation (51) in the case of the dummy reservoir.

$$Q_{i,j,c} = X_{i,j,c} \cdot TFD_i \tag{50}$$

$$Q_{i,resv,c} = \left(100 - \sum_{j=1}^{Nps-1} X_{i,j,c}\right) \cdot QTD_i \tag{51}$$

Therefore, the Equation (49) could be written again as:

$$min\, f(x)_{c,i} = \sum_{j=1}^{Nps-1} \left(Q_{i,j,c} \cdot PH_{i,j,c}\right) + Q_{i,resv,c} \cdot PH_{i,resv,c} \quad \textbf{(52)}$$

Where,

$Q_{i,j,c}$ is the flow rate to be supplied by each inflow node *j* over the period *i* within the combination *c*, and

$Q_{i,resv,c}$ is the flow rate to be supplied by the dummy reservoir over the period *i* and combination *c*.

Finally, if no difference is made between the inflow nodes and the dummy injection reservoir, the Equation (52) is expressed in its simplest form:

$$min\, f(x)_{c,i} = \sum_{j=1}^{Nps} \left(Q_{i,j,c} \cdot PH_{i,j,c}\right) \quad \textbf{(53)}$$

This function is subject to some external indirect restrictions. These restrictions are implicit in the process to compute the SC. Therefore, they do not need to be added to the OF. However, they are handy when the search space to find the optimal solution need to be limited. These restrictions are listed below:

- The sum of the percentages of the flow demand distributed among the pumping stations must meet the 100% of the total flow demand over the period *i*.

$$\sum_{i=1}^{Nps} X_{i,j,c} = 100\% \quad \textbf{(54)}$$

- The flow to be supplied by any of the pumping stations shall be between 0% and a maximum of 100% of the flow demand unless otherwise condition stated for the maximum inflow allowed in a predetermined pumping station. Thus, the search space will be restricted by physical limitations of the pumping stations and by the maximum requirements of the network:

$$0 \le x_{i,j,c} \le 100 \quad \textbf{(55)}$$

- Of course, the minimum pressure at critical node must be satisfied. Though, this condition is accomplished indirectly within the SC method calculation and always will be controlled by the adjustment of the elevation of the dummy reservoir.

$$PHc_i = PH_{min} \quad \textbf{(56)}$$

There are other constraints which are related to the hydraulic model and also must be accomplished:

a) constraints of conservation of flow and conservation of energy,
b) constraints of elevations, and
c) restrictions of non-negativity of some variables.

Some additional considerations are that the total number of combinations or inflow distributions among the pumping stations will be repeated for each period of simulation *i*. Besides, the whole flow demand in each time *i* will depend on the point of demand curve of the network at the same time.

4.2. Discrete Method

In the case of the D-M, the minimum value of the function is got after trying a finite set of combinations of the inflows at pumping stations. The inflows are calculated as a percentage of the demand. This means that, the variable of decision is given by that percentage values $X_{i,j,c}$. In that context, the D-M tries to generate a fixed number of values for the variable of the decision to be tested in the OF defined in the problem formulation. Thus, the optimal flow distribution will be found within the group of percentage values generated. Besides, it has to be considered that the same set of values is tested for each demand of the network. In this sense, the SC got for each pumping station will be approximated. This because the more accurate the calculation of the SC is, the more combinations need to be generated and tested. This becomes an even more difficult task when the number of pumping stations increases, and the number of combinations growths exponentially. Moreover, it has to be kept in mind that the number of combinations will be lower because of the restrictions of the problem (Equation 54 and 55), even so, the number of combinations can be enormous. In that way, the function has to be minimised for each period of analysis and all the possible combinations.
To know the number of combinations to be assessed, first of all, it must be defined an incremental value (Δx) of the inflow values related with the flow rates that each pumping station is able to feed into the network. Also, the number of pumping stations available to the network must be defined. For better understanding the process, the computation of the number of combinations in the case of two pumping stations (Nps = 2) will be developed. In order to do that, it will be assumed that the values of $X_{i,j,c}$ will range from 0 to 100 in increments of 10 percent ($\Delta x = 10\%$). In this way, Table 17 is constructed. It can be observed that is possible to achieve a total of 11 combinations. Besides, the distributions always sum 100%. It has to be noted that the total combinations are independent of the number of periods of analysis.

Table 17. Combinations between two pumping stations with increments of 10%

Combinations	$X_{i,1}$(%)	$X_{i,2}$ = 100%-$X_{i,1}$
1	0	100
2	10	90
3	20	80
4	30	70
5	40	60
6	50	50
7	60	40
8	70	30
9	80	20
10	90	10
11	100	0

By the same considerations made in the case of two pumping stations, a draw for several pumping stations and several increments of ΔX can be built (Figure 42). Thus, the number of evaluations required to minimize the function will be more visual. The figure shows that the number of combinations is function of the number of pumping stations and the increments of ΔX $(2\%; 5\%; 10\%; 20\%)$.

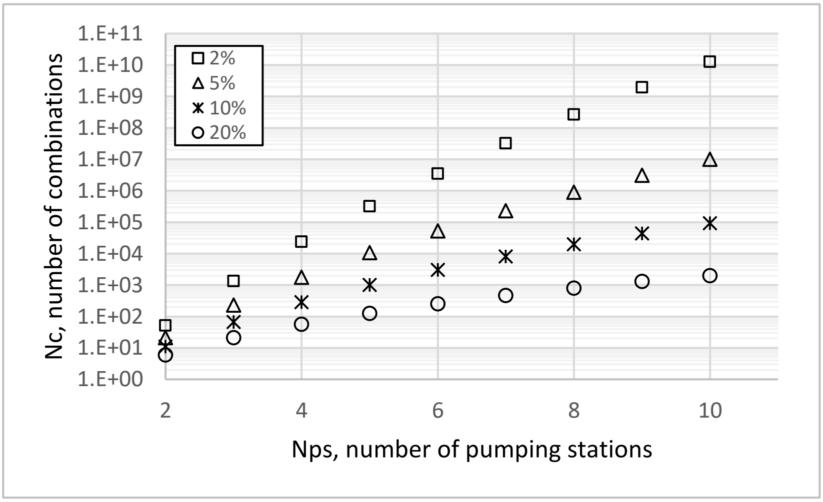

Figure 42. Combinations for several pumping stations with different values of ΔX

It is important to notice that the role of ΔX is not to increment or reduce the demand of the network but increment or reduce the inflow of pumping stations. Besides, the number of combinations implies calculations only for one stage. Moreover, the number of analysis stages will be given by the points of the demand curve of the network. Usually, one stage corresponds to a one-hour period. Therefore, the number of evaluations of the function will be the product of the number of combinations and the number of stages $(Nc \cdot Nst)$.

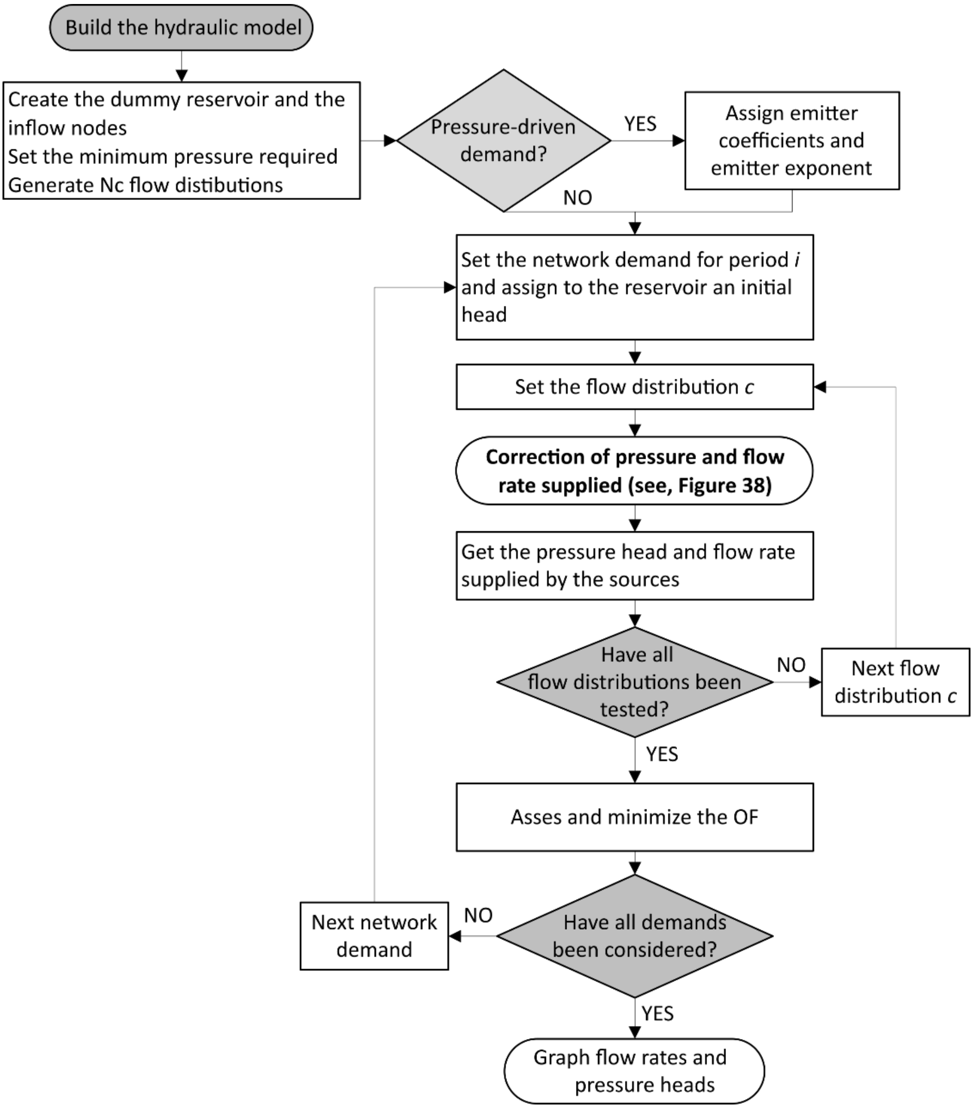

Figure 43. Diagram of the Discrete Method to calculate the optimal setpoint curves

The D-M is shown schematically in Figure 43. Since the method is based on SC calculation, one sub-process has been referred to Figure 38. This sub-process has been called *"correction of pressure and flow rate supplied"*. The subprocess consists in to get the pressure head at the critical node and the flow supplied by the sources. Then, both values are adjusted according to the methodology of the SC calculation.

From the methodology already explained to compute the SC in networks with several pumping stations and without storage capacity, some additional steps are added. Besides, pressure-driven demand (PDD) and non-pressure driven demand (NPDD) is considered. In this way, the stages of the optimisation process can be stated as follows:

1. The pumping stations that will be work as dummy reservoir must be defined (it can be any one of the available pumping stations). In the same way, the other *Nsp-1* pumping stations will be represented as inflow nodes.
2. The *Nc* combinations of flow distributions to be tested are generated.
3. If pressure-driven demand (PDD) is considered, the emitter coefficient, as well as the emitter exponent, have to be allocated to the nodes of the model. Any emitter coefficient has to be assigned to those nodes that are representing pumping stations.
4. The initial stage of demand (i.e. one point of the demand curve of the network) for the period *i* must be defined.
5. The initial elevation of the reservoir is assigned arbitrary ($Hd_{i,0,c}$).
6. Also, the combination *c* of the flow rates to be tested is assigned to the inflow nodes ($Q_{i,j,c}$). This means, the flow rate *j* for the period combination *c* is given by:

$$Q_{i,j,c} = -x_{i,j,c} \cdot TFD_i; j = 1, \dots \dots, Nps \ and \ c = 1, \dots \dots, Nc \quad \textbf{(57)}$$

   Equation (57) also has been presented in the problem formulation (Equation 50).

7. The next step consists in the SC calculation process application (see, sections 3.3. and 3.4.). The steps are the following:

   - Once input information of the model has been set, the hydraulic analysis is run aiming to get the pressure head $\left(PH_{r,c}\right)$ for the total number of demand nodes (*TN*). Then the minimum pressure head at the critical node can be determined ($PHc_{i,c}$).

$$PHc_{i,c} = min\left(PH_{r,c}\right); r = 1, \dots \dots, TN \ y \ c = 1, \dots \dots, Nc \quad \textbf{(58)}$$

   - Then, taking as reference the value of the minimum pressure head required in the network (PH_{min}), the deficit or excess of pressure head at critical node for stage *(i)* and combination *(c)* is calculated.

$$\Delta PH_{i,c} = PHc_{i,c} - \ PH_{min}; i = 1, \dots \dots, Nst \ and \ c = 1, \dots \dots, Nc \quad \textbf{(59)}$$

   - Next, the elevation of the dummy reservoir is corrected.

$$Hd_{i,c} = Hd_{i,0,c} + \Delta PH_{i,c} \quad \textbf{(60)}$$

- Then, the new elevation is assigned to the reservoir, and further analysis is performed. The iterations end when the value of the minimum pressure head requires is the same as the pressure head at the critical node.

$$Hd_{i,0,c} \approx Hd_{i,c} \quad \textbf{(61)}$$

- When using PDD, before a new analysis it is required to recalculate the consumption of the network since it will change according to the pressure head at nodes. Thus, the flow rate distributions among pumping stations are also repeated. Then, the process is repeated from step 7.
- At the moment that the minimum pressure head condition is reached, it must be checked if the flow rated feed is the same as the demand in the network. Otherwise, the flow rate distribution is done again until meeting this second condition.

$$\sum_{j=1}^{Nps} \left(Q_{i,j,c}\right) = TFD_i \quad \textbf{(62)}$$

8. Once the pressure head and flow rate conditions have been met, the next step is determining the flow rates and pressure heads of each pumping station ($Q_{i,j,c}; PH_{i,j,c}$). It is important do not forget the elevation of the reservoir corresponds to the HGL elevation at the pumping station discharge represented as a dummy reservoir. Therefore, the pressure head of the dummy reservoir ($PH_{i,resv,c}$) has to be calculated subtracting the HGL elevation at the suction (Hs_i) from the HGL elevation at the discharge ($Hd_{i,c}$).

$$PH_{i,resv,c} = Hd_{i,c} - Hs_i \quad \textbf{(63)}$$

9. Then, the OF is assessed for the period of analysis *i* and combination *c*. This value has to be recorded to be compared at the end with the other values resulting from assessing the function with the remaining number of combinations.

$$f(x)_{c,i} = \sum_{j=1}^{Nps} \left(Q_{i,j,c} \cdot PH_{i,j,c}\right) \quad \textbf{(64)}$$

i = 1, ……, Nst; j = 1,......, Nps-1; c = 1, ……, Nc

10. After finishing the analysis for the first combination, the process is repeated until all the *Nc* combinations are assessed.

11. Finally, the best distribution of flow rates is got after finding the minimum value of the function from all the *Nc* values of the function that have been obtained for the period of analysis *i*.

$$Opt_i = min\left[F(x)_{i,c}\right] \quad \textbf{(65)}$$

12. All the steps are repeated for each stage or period *i*. Thus, the outputs of the problem are *Nc* SCs, one for each combination and for each pumping station j. The SCs will have as many points as analysis scenarios; this means one point for each period of simulation. After the optimisation is developed, it will be got just one SC for each pumping station. The optimal Sc of each pumping stations will be composed by the optimum flow rate distributions for each period of analysis.

4.3. Continuous Method

In the D-M, the evaluation of the function presented in the problem formulation implies the use of a finite set of *Nc* combinations (i.e. flow distributions). These *Nc* combinations are increased according to the number of pumping stations and the accuracy desirable to find the optimal distribution of flow rates as well as compute the optimal SCs. However, the C-M aims to consider the flow supplied by the pumping stations as continuous variables. Thus, the decision variable is the same ($X_{i,j}$) but its definition is not subjected to a number of combinations. In that sense, the problem formulation is still valid (see, section 4.1.) but the combinations of flow distributions will be given by the search algorithm. Therefore, it is not necessary to construct a finite set of distributions of the injected flow rates to find the optimum, since search will depend on the number of evaluations of the OF performed by the search algorithm.

It is essential to select the algorithms that better fit the problem [50]. In this work, two algorithms have been chosen with the goal of contrast results: Hooke and Jeeves algorithm (H-J) and Nelder and Mead algorithm (N-M). Both accomplish with the following characteristics:

a) Function derivatives are not needed,
b) multidimensional searching, and
c) certain types of constraints are permitted.

The methodology to be applied is quite like the D-M. Nevertheless, some steps that depend on the search algorithm will be added, and there are others that will be removed as it can be noted in the description of the section below. First, the optimisation process will be described in function of the algorithm H-J and then regarding N-M algorithm.

4.3.1. Application of search algorithm Hooke and Jeeves

Both, the SC calculation methodology and the H-J algorithm have been already presented separately (see, section 2.3.1. and chapter 3). However, the C-M is based on the altogether application of both of them. A general scheme of the process can be appreciated in Figure 44. Since the figure is simplified, the H-J algorithm movements are only stated. Thus, for a more detailed explanation, the section of H-J algorithm in Chapter 2 has to be revised. Moreover, previous to the OF assessment, the sub-process *"pressure head and flow computation"* must be done. The sub-process has been presented in Figure 38.

The steps of the process are presented next:

1. The first step is defining the parameters of the H-J algorithm. Based on the recommendations of the literature, the values shown in Table 18 will be assumed.

Table 18. Parameters for Hooke and Jeeves algorithm

Parameters	Value	Description
F(X)	-	Target function to be minimized
Nps	-	Number of pumping stations available on the network
E	0.001	Stop control parameter
D	0.1	Length of the search step
$\vec{X}_0 = X_i, \ldots, XNps$	(0,0, 0, …, 100)	Starting point. Although it can be arbitrary selected, for the cases of study, the 100% of the flow distribution will be assigned to the dummy reservoir initially.

2. The information necessary to assess the OF also has to be defined (Table 19).

Table 19. Additional information to assess the objective function

Parameter	Description
$Hd_{i,0}$	HGL elevation at discharge of the pumping stations represented as a dummy reservoir
Hs_i	HGL elevation at suction of the dummy reservoir
PH_{min}	Minimum pressure head requirement

3. Depending on the number of pumping stations, a starting search point will be assumed as H-J algorithm requires.
4. If PDD are considered, emitter coefficients and the emitter exponent must be defined at nodes.

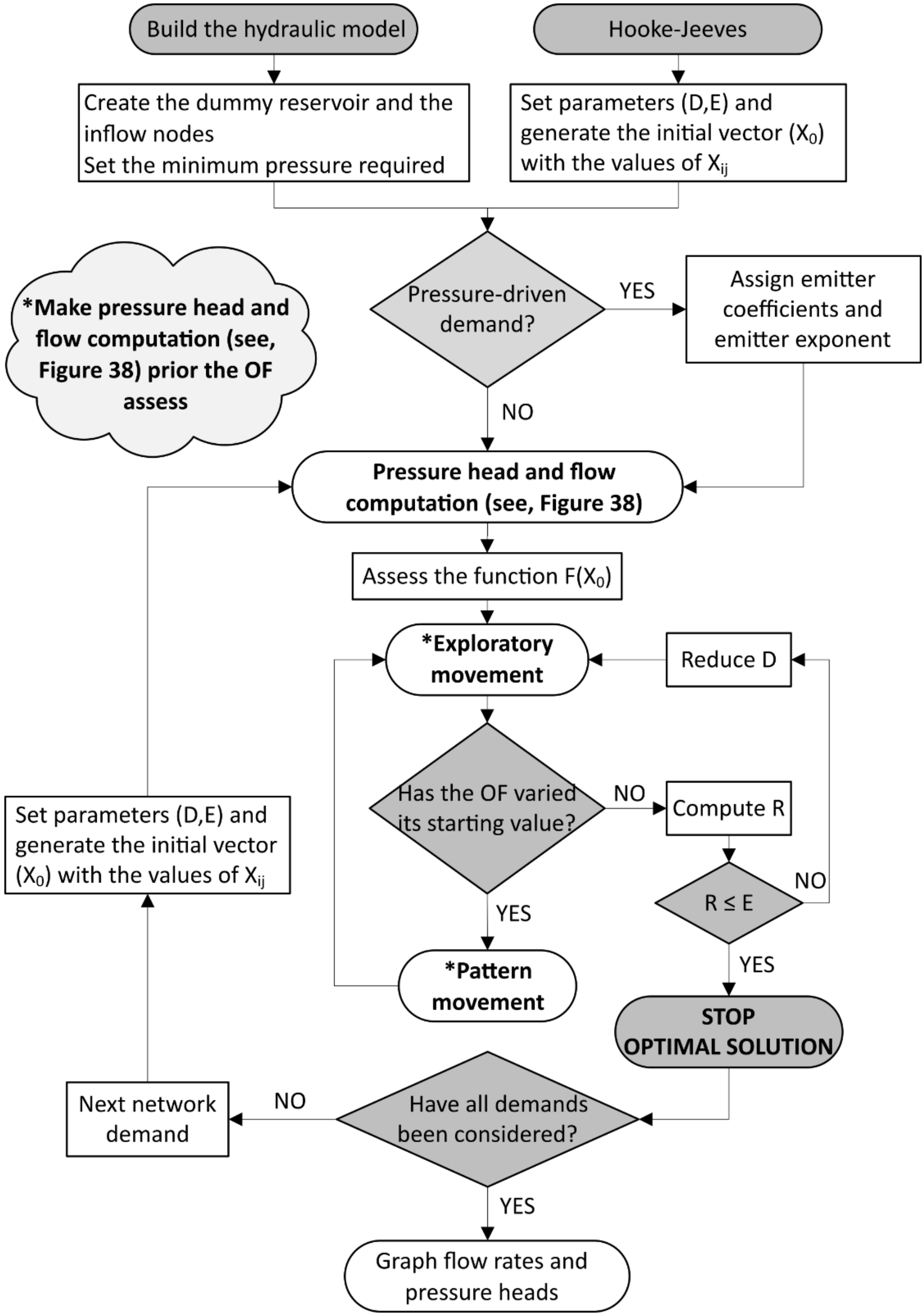

Figure 44. Optimum flow distribution by means of Hooke and Jeeves algorithm

5. The number of stages of analysis must be defined. If the network has a curve of demand, the number of stages will be determined by the number of points of the curve. Besides, an analysis of an established range of flow demands could be carried out. This range could be defined by a minimum demand, a maximum demand and an incremental value of the demand.
6. The OF has to be assessed for the initial search point (Equation 64). Each time that function is evaluated, the next steps have to be followed:
   - Define the total flow demand of the network.
   - Allocate the flow rate distribution among the inflow nodes defined by H-J algorithm (Equation 57).
   - Determine the critical node of the network and its pressure head (Equation 58).
   - Calculate the deficit or excess of pressure head at critical node (Equation 59).
   - Correct the HGL elevation of the dummy reservoir.
   - Perform a new analysis until the difference between the minimum pressure needed and the minimum pressure at the critical node is minimum or null.
   - Check the flow rates provided by the pumping stations are the same as the demand in the network (Equation 62) and perform a new analysis if it is required.
   - Record the pressure heads and flow rates from each pumping station.
   - Compute the value of the OF.
7. The exploratory movement of the optimisation algorithm is initialised, for which an additional restriction is added (Equation 55). The restriction limits the search space of the flow distributions in a certain way. Thus, all solutions where the inflow of pumping stations exceed the 100% or is under the 0% of the total flow demand are neglected.
8. Each time a new exploratory movement is carried out, the OF is re-evaluated.
9. Once all dimensions have been explored, i.e. the distribution of flow rates, it must be decided if the pattern movement is activated. Otherwise, the exploratory action is performed once more time but changing the length of the search step. Either of the choices will depend on whether a better value of the OF has been found.
10. Whether the pattern movement is activated, a new search point will be created. Then, the OF will be re-evaluated. Depending on if the new search point does not produce better results than the previous point, the exploratory movement will start again taking as a reference the last best point.
11. The process is iterative and is developed until it is not possible to generate better search points. In that case, the stop criterion must be checked. Otherwise, the search keeps going but with a different length step.
12. The best combination of flow rates distribution and pressure heads are recorded, of the corresponding pumping stations.

13. The methodology is developed for each stage; this is for each change of the network demand.
14. Finally, the graphics of the SCs are got.

4.3.2. Application of search algorithm Nelder and Mead

It has been already mentioned that N-M algorithm has the role of a second search algorithm that contrasts the results got by H-J algorithm. This is a measure to assure that the optimal solution is calculated. Thus, at the end, the results of both algorithms are compared. A simplified scheme of the optimisation applying both N-M and SC is presented in Figure 45. The figure states the movements of the N-M algorithm (see, section 2.3.2.) and the sub-process of *"pressure head and flow computation"*. The sub-process is part of the SC calculation methodology and has been already presented in Figure 38. The optimisation process that integrates the N-M algorithm is described below.

1. First of all, the parameters of the algorithm must be defined. For that, it will be assumed the values of Table 3. Besides, the parameters of the hydraulic model also have to be set (Table 19). The algorithm will stop when the difference between averages values of the function is lower than a certain tolerance *E*, in this case $E = 10^{-10}$
2. It will be created *Nps +1* initial solution vectors or vectors of flow distributions $(\overrightarrow{X_0}_j, \overrightarrow{X_0}_{j+1}, \dots, \overrightarrow{X_0}_{Nps+1})$, each one with *Nps* dimensions.
3. Though the vectors are random, each time a new vector is generated care should be taken to keep their values between the range of zero and 100% (Equation 55).
4. The OF is evaluated, and the results are sorted from lower to higher $F(\overrightarrow{X_0}_1) \leq, \dots, F\left(\overrightarrow{X_0}_{Nps+1}\right)$.
5. In the same way as with H-J algorithm, each time that OF is assessed, the next steps must be followed:

   - Define the total flow demand of the network.
   - Allocate the flow rate distribution among the inflow nodes defined by N-M algorithm (Equation 57).
   - Determine the critical node of the network as its pressure head value (Equation 58).
   - Calculate the deficit or excess of pressure head at critical node (Equation 59).
   - Correct the HGL elevation of the dummy reservoir.
   - Perform a new analysis until the difference between the minimum pressure need and the minimum pressure at the critical node is minimum or null.
   - Check that the flow rates provided by the pumping stations are the same as the demand for the network (Equation 62) and perform a new analysis if it is required.

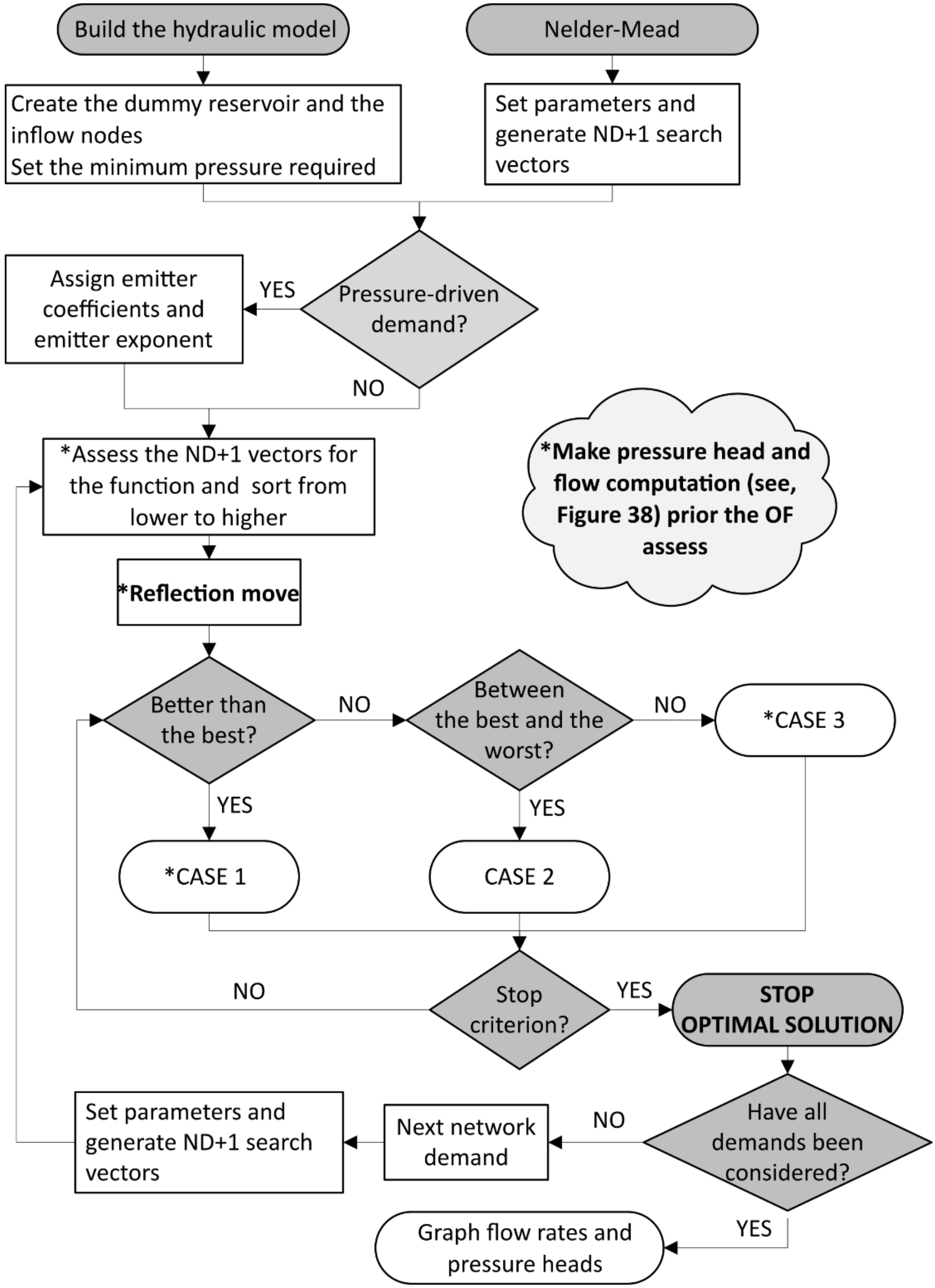

Figure 45. Optimum flow distribution by means of Nelder and Mead algorithm

- Record the pressure heads and flows from each pumping station.
- Compute the value of the OF.

6. The reflection vector $(\overrightarrow{X_R})$ is obtained (Equation 10) as well as the value of the function $F(\overrightarrow{X_R})$.
7. If $F(\overrightarrow{X_R}) < F(\overrightarrow{X_{0_1}})$, the expansion vector $(\overrightarrow{X_E})$ must be found (Equation 11):

   a) If $F(\overrightarrow{X_E}) < F(\overrightarrow{X_R})$ then $F\left(\overrightarrow{X_{0_{Nps+1}}}\right) = (\overrightarrow{X_R})$. The iteration finishes and a new one starts.

   b) If $F(\overrightarrow{X_E}) \geq F(\overrightarrow{X_R})$ then $F\left(\overrightarrow{X_{0_{Nps+1}}}\right) = (\overrightarrow{X_E})$. The iteration finishes and a new one starts.

8. If $F(\overrightarrow{X_{0_1}}) \leq F(\overrightarrow{X_R}) < F\left(\overrightarrow{X_{0_{Nps+1}}}\right)$ then $F\left(\overrightarrow{X_{0_{Nps+1}}}\right) = (\overrightarrow{X_R})$. The iteration finishes and a new one starts.
9. If $F(\overrightarrow{X_R}) \geq F\left(\overrightarrow{X_{0_{Nps}}}\right)$ then

   a) If $F(\overrightarrow{X_R}) < F\left(\overrightarrow{X_{0_{Nps+1}}}\right)$ then outward contraction vector $(\overrightarrow{X_{CO}})$ is got (Equation 12).

   - If $F(\overrightarrow{X_{CO}}) < F(\overrightarrow{X_R})$ then $F\left(\overrightarrow{X_{0_{Nps+1}}}\right) = F(\overrightarrow{X_{CO}})$. The iteration finishes and a new one starts.
   - If $F(\overrightarrow{X_{CO}}) \geq F(\overrightarrow{X_R})$ then shrink movement starts.

   b) If $F(\overrightarrow{X_R}) \geq F\left(\overrightarrow{X_{0_{Nps+1}}}\right)$ then inward contraction vector $(\overrightarrow{X_{CI}})$ is calculated (Equation 13).

   - If $(\overrightarrow{X_{CI}}) < F(\vec{X}_{Nps+1})$ then $F\left(\overrightarrow{X_{0_{Nps+1}}}\right) = F(\overrightarrow{X_{CI}})$. The iteration finishes and a new one starts.
   - If $(\overrightarrow{X_{CI}}) \geq F(\vec{X}_{Nps+1})$ then *shrink movement* starts.

10. In the case that *shrink movement* is initialised, *Nps* new vectors are created (Equation 14). Thus, the next simplex is formed by the previous best point and the *Nps* new vectors $(\overrightarrow{X_{0_1}}, \overrightarrow{V_2}, \ldots, \vec{V}_{Nps+1})$. Then, the iteration finishes and a new one starts.
11. Each time a new simplex is formed a further analysis is performed until reach the stop criterion. When the stop criterion is reached the process is repeated for the next flow rate demand of the network up to all *Nst* stages have been considered.
12. Finally, the graphics of the SCs are got.

4.4. Sensitivity of the flow distribution among the pumping stations of a network

For a better understanding of the variables that influence the optimum flow distribution of the discharge amount the pumping stations, it is proposed a simple system that will be modelled in EPANET. Two pumping stations (PS1 and PS2) are available (Figure 46). One of them (*PS1*), will be represented as a dummy reservoir and the other one as inflow node. Also, there are only two demand nodes. The baseline stage consists of two pumping stations that have the same elevation and are equidistant to the nodes of consumption. Also, the nodes have the same both the demand and elevation.

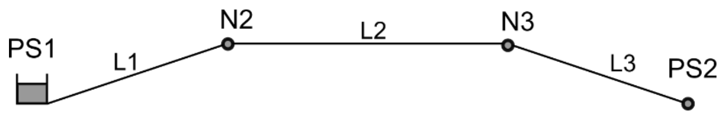

Figure 46. Test network for the sensitivity analysis of the flow distribution

The head loss will be computed using the Darcy-Weisbach equation. The roughness of the pipes is 0.1. The information of both the pipelines and junctions are presented in Table 20 and Table 21 respectively.

Table 20. Pipelines information of the test network.

ID Pipelines	Length	Diameter	Roughness
	m	mm	Mm
1	1000	260	0.1
2	5000	260	0.1
3	1000	260	0.1

Table 21. Junctions information of the test network

ID Junctions	Elevation	Base Demand
	m	LPS
2	10	50
3	10	50
PS 2	0	0
Reservoir PS1	0	-

The variables that will be part of the sensitivity analysis are the following:

a) Pipelines length
b) Flow rate demands

c) Elevation of the consumption nodes
d) Elevation of the pumping stations
e) Minimum pressure required
f) Roughness
g) Diameter

The sensitivity analysis will be developed applying the D-M, which has been programmed in Microsoft Visual Studio 2010. The demand curve will be given in Table 22. The minimum pressure required is 35 m. Also, the flow rates of discharge of pumping stations will be analysed for distributions with increments of 10% as it was shown in Table 17.

Table 22. Demand curve for the test network

Period (*i*)	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
Demand Multiplier	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1

The sensitivity analysis will be developed for NPDD. The influence of the PDD will be considered in the cases of study, later presented.

As it has been mentioned before, there will be as many SCs for each pumping station as combinations of flow distributions between the two pumping stations are considered. In that context, though all of them are drawn, it will not be possible visualise the optimal flow distribution. Thus, it is required to generate a different type of graphic. In this case, the total energy that is necessary to satisfy both the demand and minimum pressure head according to the flow distribution will be drawn. For instance, if the demand is 10 l/s there will be eleven possible distributions between pumping stations by Table 17 to meet the demand. So, there will also be eleven values of the total energy corresponding to each one of the possible distributions available. However, there is only one flow distribution that minimises the use of energy. Thus, if the minimum value divides all the energy values, the unitary energy corresponding to each distribution will be obtained. In this way, the unit value will correspond to the optimum distribution. In the case of 10 l/s, the optimal allocation is 50% for the pumping station PS1 (5 l/s) and 50% for the pumping station PS2 (5 l/s). The same can be done for all demands, in this way it will be possible to visualise the optimal distribution for each one of them. Thus, Figure 47 shows the total unitary energy needs of the pumps system when pumping station PS1 assumes a percentage of the demand. This, for each period of analysis. It can be appreciated that in all the cases of the network demand, the minimum energy is reached when the inflow of pumping station PS1 is 50%. This is quite reasonable since both pumping stations are equidistant. Therefore, there is no reason to think that one of them will need more energy than the other.

On the other hand, the graphic will be the same in the case of pumping stations PS2, although the values of the x-coordinate axis will be arranged in descending order. That is, when the percentage of the demand assumed by the PS1 station is 0%, that of the PS2 station will be 100% and so on for all other values.

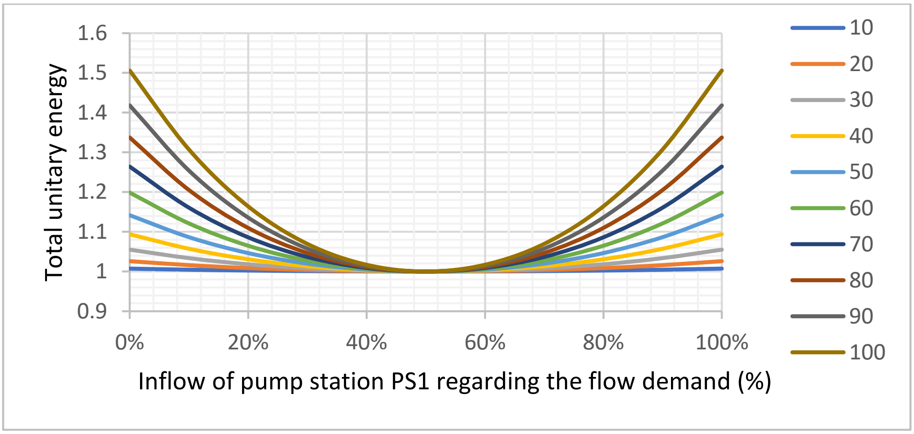

Figure 47. Inflow flow rate percentage of pumping station PS1 vs total unitary energy

The optimal flow distribution depending on the variation of the demand is shown in Figure 48. The figure shows with more accuracy than the optimal flow distribution corresponds to an allocation of 50% between the pumping stations independently of the network demand. Starting from that, it is possible to draw the optimal SC that requires the minimum needs of energy.

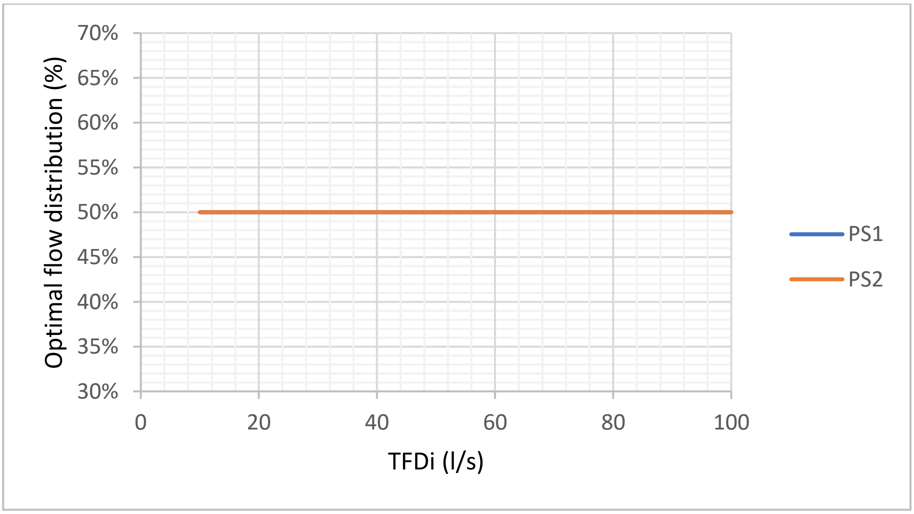

Figure 48. Optimal flow distribution between pumping stations PS1 and PS2

Since the optimal distribution remains constant, the SC is the same for both pumping stations, and the range of flow supplied goes from 5 l/s to 50 l/s according to the optimal flow distribution (Figure 49).

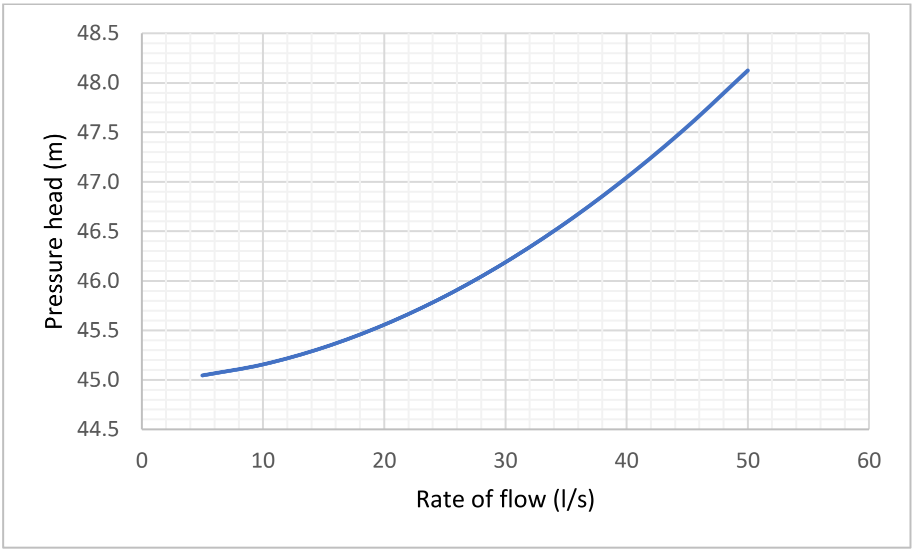

Figure 49. Optimal setpoint curve

4.4.1. Pipelines length

The modification of the lengths of the pipes involves considering two cases:

a) The increase or decrease in the distance between points of consumption.
b) The increase or decrease of the distance between the pumping stations concerning the nodes of consumption.

According to the first case, the length of the pipeline 2 (Figure 46) should be increased or decreased. Nevertheless, the optimal flow distribution will be the same that the base case since both pumping stations will remain equidistant. Hence, the analysis will be performed only for the second instance. To do that, four-length values of the pipeline 3 have been considered: 2000 m, 2500 m, 3000 m and 5000 m. When the length is 2000 m, the optimal flow distribution is the same as the base stage (Figure 48). However, for the rest of the cases, the optimal flow distribution was PS1= 60 % (Figure 50) and PS2 = 40% (Figure 51). Though the values of the lines of the energy are different, only will be presented the corresponding to the length of 5000 m.

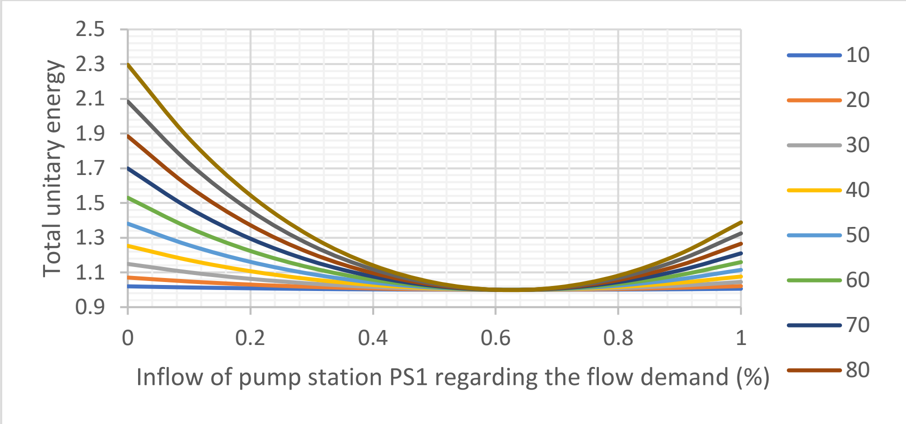

Figure 50. Inflow flow rate percentage of pumping station PS1 vs total unitary energy (Length = 5000 m)

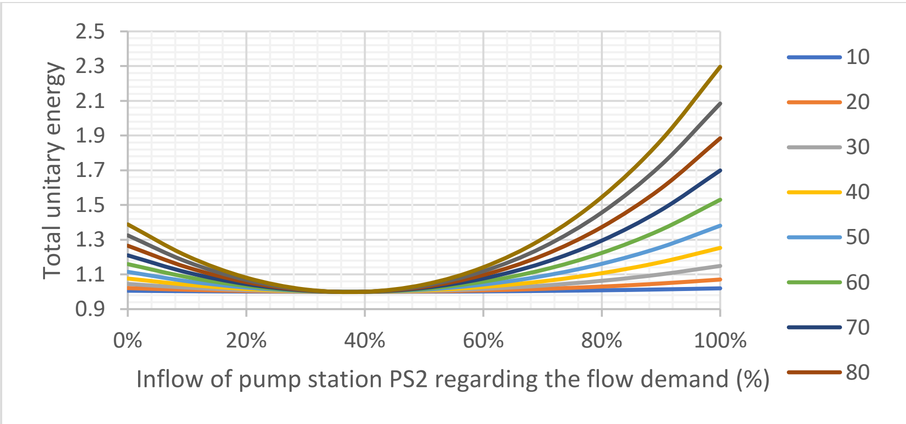

Figure 51. Inflow flow rate percentage of pumping station PS2 vs total unitary energy (Length = 5000 m)

The optimal flow distribution remains constant for all the demanded flows as can be noted in Figure 52. It can be observed that a higher percentage of the flow distribution is assigned to pumping station PS2, since it is closer to the network and consume lower energy than pumping station PS1. Apparently, the losses to be overcome are more moderate and therefore less power is needed. Although the influence of the location of the source is indeed a factor to consider, a considerable variation of the distance to the pumping station has to occur before there was a variation of the optimal distribution.

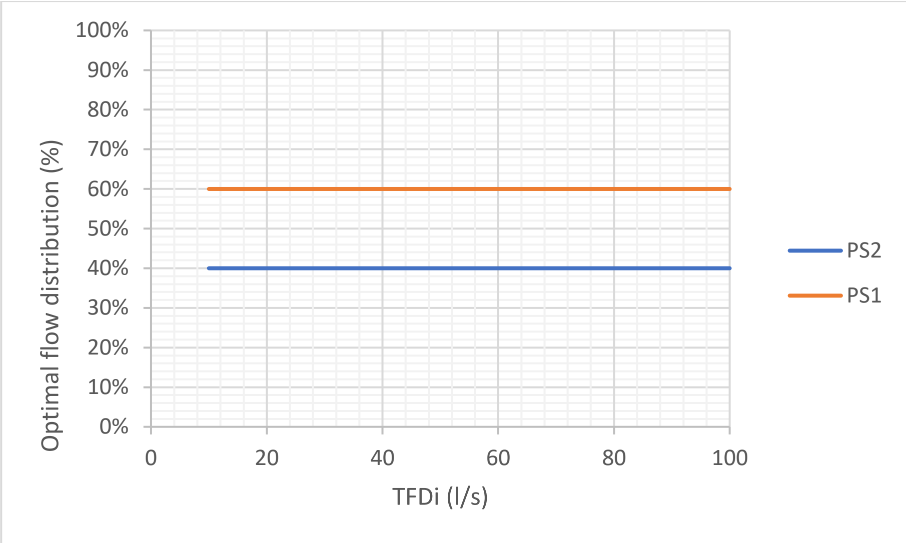

Figure 52. Optimal flow distribution between pumping stations PS1 and PS2 (L = 2500 m, 3000 m and 5000 m)

As the optimal distribution changes, the SCs also change. The SCs will be different in each case when the length change. As an example, only those corresponding to a length of 5000 m are presented. In this way, the new optimal SCs are given by the next figure.

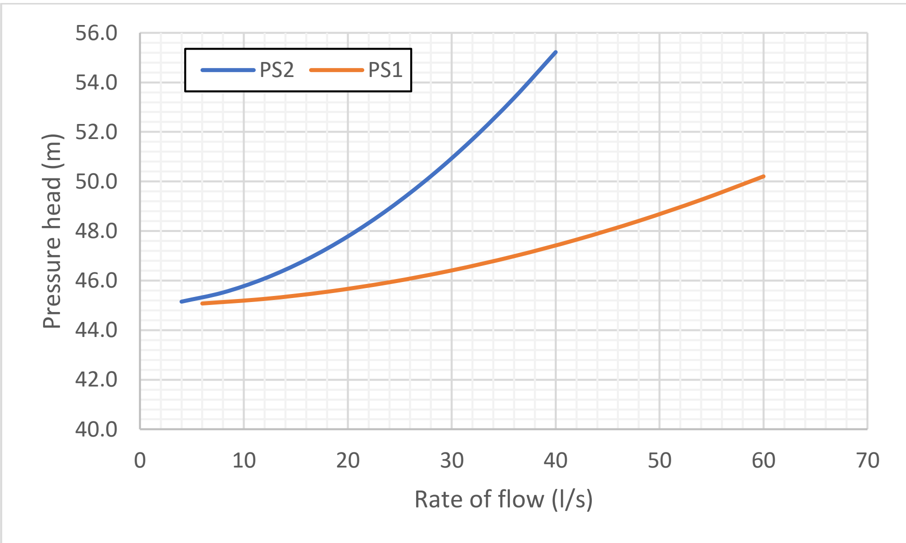

Figure 53. Optimal Setpoint curves when L = 5000 m

4.4.2. Flow demands

For the analysis of the demand, the consumption of the node 3 will be changed. For that, it is established that the variation of the demand will be in the range of 0 to 200% of the base demand. Indeed, this kind of difference is not common, but it will be useful for showing the sensitivity of the flow distribution to the changes of the demand.

When the node 3 has null demand, the optimal flow distribution is not the same as the base stage, but it is still constant over the whole range of the demands of the network (Figure 54). In this case, pumping stations PS1 should provide 70% of the demand, and pumping stations PS2 the 30%. As it happens with the variation of lengths, pumping station that is closer to the point of the demand (i.e. the node 2) assumes a higher percentage of the distribution.

For the case of an increase of a 100% of the demand at node 3, the demand of that node will be higher than the node 2. Thus, pumping station PS2 should supply a 60% of the total demand as it has a more benefit location and pumping station PS1 the remaining 40% (Figure 55).

When the demand of node 3 rises a 200%, the optimal flow distribution is the same as Figure 55. Thus, it can be thought that despite that nodes can follow different demand curves, for small variations is not expected that the optimal distribution change too much.

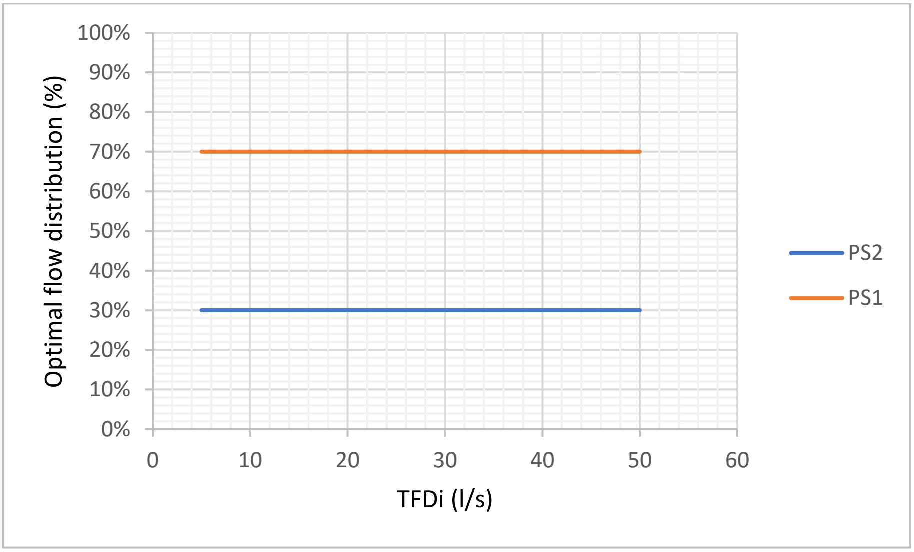

Figure 54. Optimal flow distribution when demand at node 3 = 0 l/s

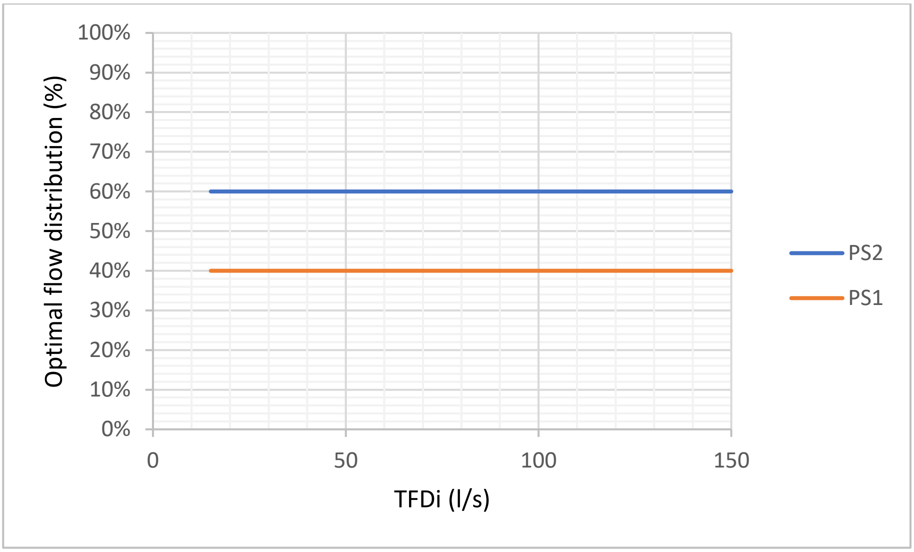

Figure 55. Optimal flow distribution when demand at node 3 increase 100%

4.4.3. Nodes elevation

Regarding the variation of the elevations two cases could be assessed:

a) The elevation variation of the consumption nodes, and
b) The elevation variation of the pumping stations.

Admittedly, a variation in the elevation of the consumption nodes is quite improbable. Although, it is feasible to think that the critical node of the network changes, this occurs in networks that are relatively flat. Thus, the analysis will be performed from the point of view that the elevation of the pumping stations can change, or at least the energy head that can be supplied. In that sense, it will be considered that the HGL elevation at the suction on pumping station PS1 can take three values: 5 m, 10 m and 45 m.

When elevation is 5 m, it can be observed that the optimal flow distribution is not constant as in the previous cases. Thus, the minimum energy for both demands 10 l/s and 20 l/s is achieved only with one pumping station (PS1) since it supplies 100% of the demand (Figure 56). On the opposite, pumping station PS2 provides 0% of the demand (Figure 57). This happens because of the additional energy that PS1 has. Therefore, a least for low values of the demand, the network does not need more than one pumping station.

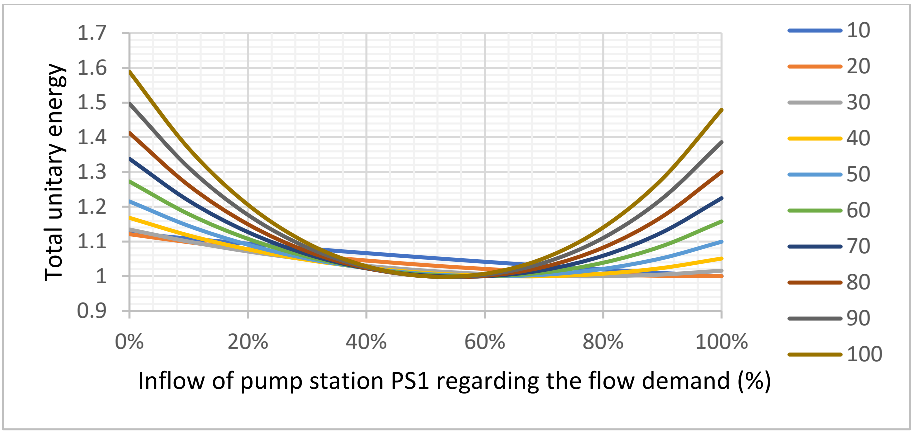

Figure 56. Inflow flow rate percentage of pumping station PS1 vs total unitary energy (Elevation = 5 m)

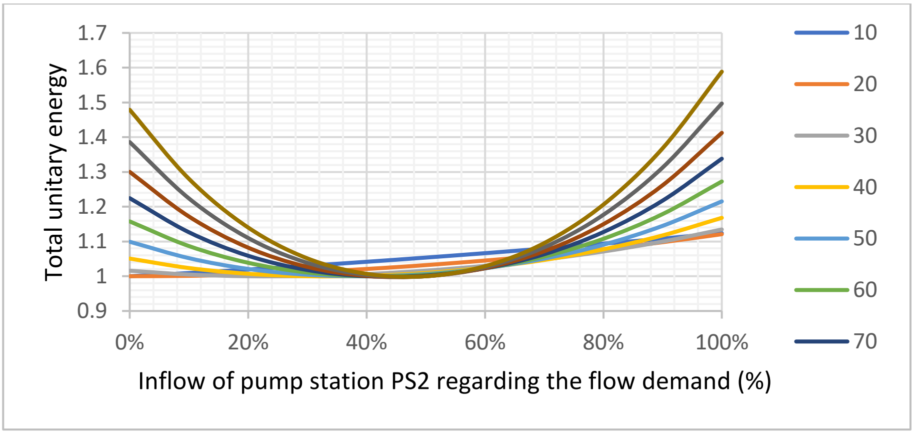

Figure 57. Inflow flow rate percentage of pumping station PS2 vs total unitary energy (PS1 elevation = 5 m)

Graphics (Figure 56, Figure 57) show that likely the optimal distribution for most of the demand is between 40% and 50% of the demand. However, if the optimal flow distribution in function of the demand is drawn (Figure 58), it can be noted that the flow distribution change over most of the values of demand. Besides, it can be observed that pumping station PS1 has a more critical role than PS2 until the amount of the demand rises 80 l/s, where the optimal flow distribution is 50% for both of them. In that context and since an energy point of view it is cheaper allocate a higher percentage of the inflow to pumping station PS1.

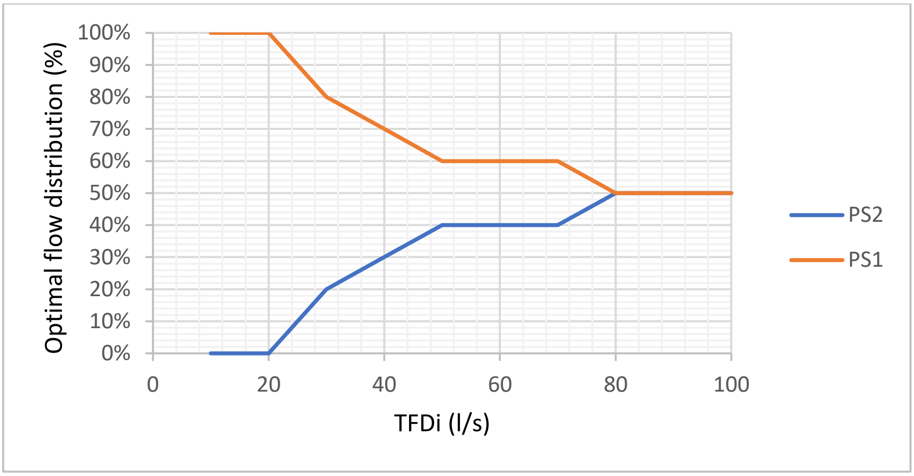

Figure 58. Optimal flow distribution when HGL elevation at suction node of PS1 = 5 m

It has to be remembered that the optimisation methodology leads to the optimal flow distribution for each value of the demand. In that context, the optimal SCs are not always uniform as Figure 53. In that way, if optimal SCs that correspond to the optimal flow distribution of Figure 58 are drawn, Figure 59 is obtained. Thus, it can be noted that in the case of the SC points of pumping station PS1, is more explicit that points of different SCs are taken to form the optimal SC. In that sense, the final SC that should follow the pumping system must be corrected to get a softer curve. Beyond, it is important to highlight that as a result of the additional energy that PS1 has, its SC shows a lower pressure head to supply the same values of flow rate that PS1.

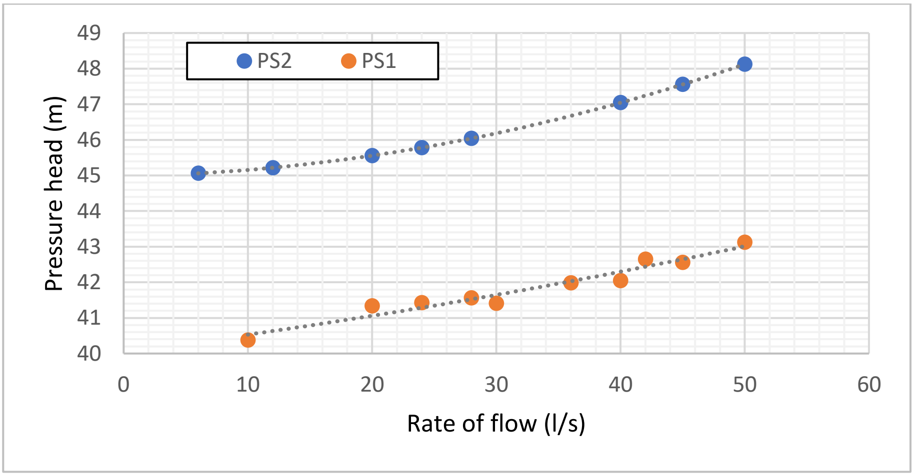

Figure 59. Setpoint curves when HGL elevation at suction node of PS1 = 5 m

If a suction elevation of 10 m (Figure 60) and then an elevation of 45 m (Figure 61) is assigned to the dummy reservoir (i.e. pumping station PS1), it can be appreciated how PS1 increase the percentage of the distribution that can supply. Of course, the energy at suction is free energy. Otherwise, the higher rate of the flow distribution would be in charge of PS2.

Of the results, it can be deducted that elevation both consumption nodes or pumping stations is a very sensitivity variable since it directly influences over the pressure head of the critical node and the energy of the system. Therefore, if there are too many variations at HGL suction elevations, it will be expected SCs with very variable points.

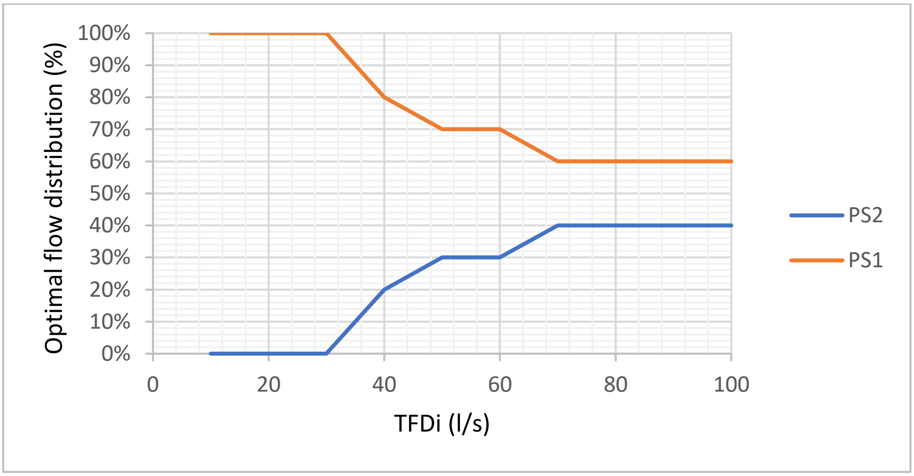

Figure 60. Optimal flow distribution when PS1 elevation = 10 m

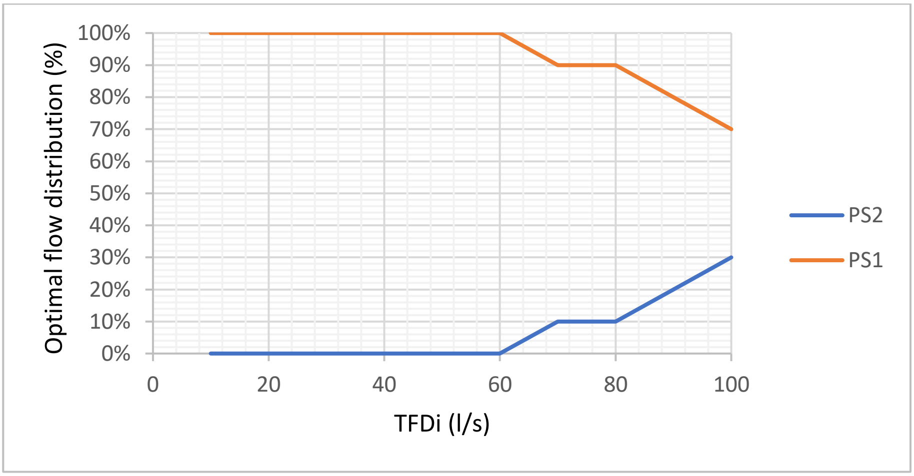

Figure 61. Optimal flow distribution when PS1 elevation = 45 m

It is important to mention that changes in the optimal distribution may seem a bit brusque. This happens because of the rounding errors as a result of the D-M aplication. Therefore, in the case of the C-M, these changes may be presented more smoothly.

4.4.4. *Minimum pressure head required*

With the aim of assessing the influence of the minimum pressure allowed in the network, three values of the pressure were tested; 25 m, 30 m and 40 m. Results show that the optimal flow distribution is the same as Figure 48. This, because of the pumping stations increase or decrease the energy uniformly to meet the pressure head requirement. Therefore, this variable does not influence in a significative way the optimal flow distribution, though it does in the SC (Figure 62).

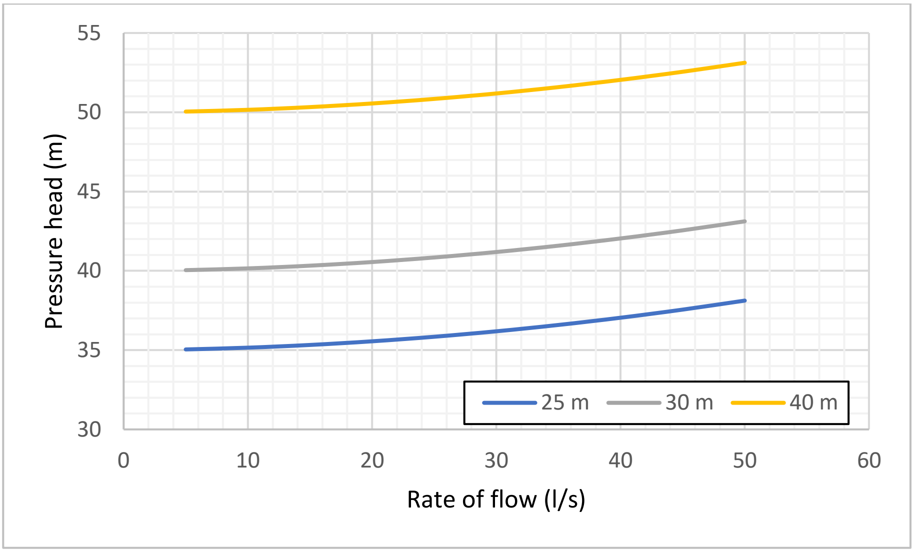

Figure 62. Setpoint curves for PS1 and PS2 for different minimum pressure head requirements

4.4.5. *Roughness*

Two scenarios were taking into account:

a) A different roughness for each pipeline, and
b) A roughness of 0.15 for all the pipelines

For each one of the cases, the optimal flow distribution was the same as the base case. Therefore, this variable is not significant enough to change the optimal flow distribution. This state will be true as long as the consumptions do not depend on the pressure. Otherwise, the head losses could be big enough to influence the flow distribution.

4.4.6. Diameter

To carry out the sensitivity analysis due to the variation of the diameter, the following cases are formulated:

a) Changing the diameter of all pipelines at the same time.
b) Modifying the diameter of pipelines that joining the consumption nodes.
c) Modifying the diameter of pipes that joining the pumping stations with consumption nodes.

If the same value changes all the diameters, it will happen the same that in the case of the minimum pressure head allowed. This means, the optimal flow distribution will be quite like the base stage. Thus, the analysis will be focused on the other two cases. In both cases, a bigger and smaller diameter than the currently installed will be applied.

If the diameter of the pipeline 2 is changed first by a value of 300 mm and then by 210 mm the optimal flow distribution is the same as Figure 48. The reason is that since the network is balanced, even if pipeline 2 is deleted the optimal flow distribution will remain constant.

For the next case, the diameter of pipeline 3 was changed to 210 mm (Figure 63). It can be observed that the optimal flow distribution is altered since pumping station PS1 provides a higher percentage of the flow distribution. The reason is that when the diameter of line 3 decrease head loss increase. Hence, less energy is needed when more flow rate is supplied through pumping station PS1.

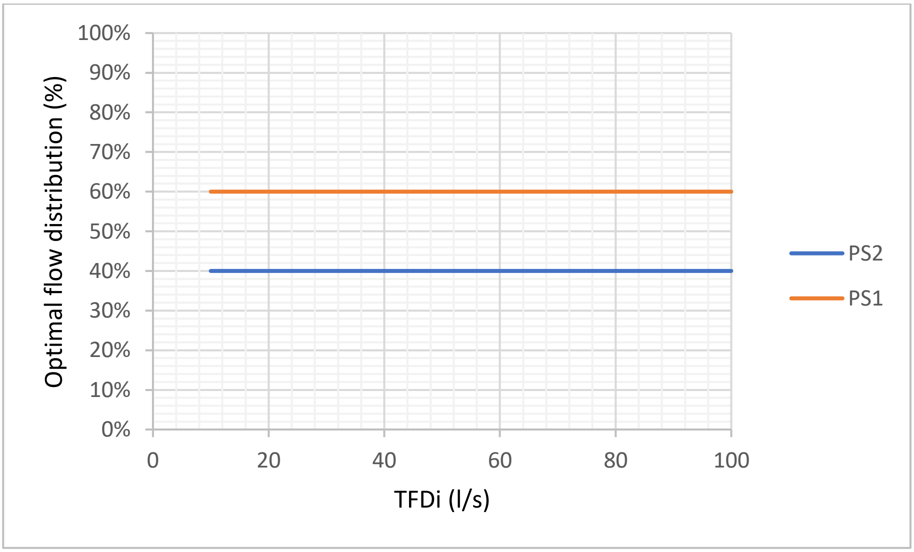

Figure 63. Optimal flow distribution when pipeline 3 diameter = 210 mm

When pipeline 3 take a diameter of 300 mm, the optimal flow distribution is the same as the base case. Therefore it has been assumed a diameter of 350 mm. It can be noted that the optimal flow distribution remains constant until demand rises 50 l/s then pumping station PS2 assumes the 60% of the demand since the head losses at pipeline 3 are lower (Figure 64). Thus, the flow distribution is influenced when the head losses at pipeline take a more important role, i.e. when the demand increase.

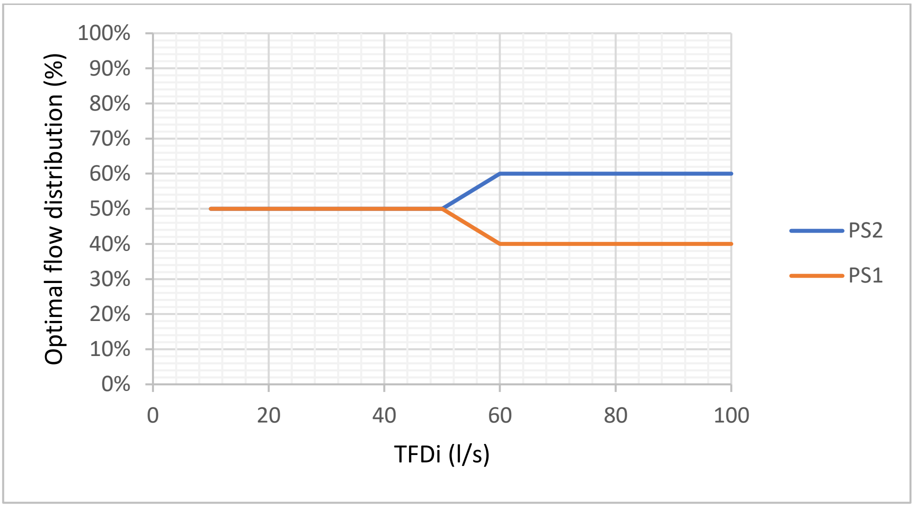

Figure 64. Optimal flow distribution when pipeline 3 diameter = 350 mm

4.4.7. Key findings of the sensitivity analysis

The sensitivity analysis can be described in accordance with the two variables of the SCs, the flow and the pressure head provided by the pumping stations. Thus, the key findings can be summarized as follows:

- The optimal flow distribution is mostly influenced by the variation of the HGL elevation at the suction node of pumping stations. In that sense, even small variations may lead to changes in the optimal flow distribution. This is, one pumping station can be preferred over another one to supply more or less flow into the network.
- Usually, the order of magnitude of the terms that do not depend on the flow (i.e. static lift, minimum pressure required) is higher than the flow rate. Therefore, for small variations of the network demand, the optimal flow distribution will remain constant.
- As the network demand increases head losses become significant. That is, the resistance generated by the network elements (diameter of pipes, length of pipes, roughness, vales, etc.) raises. Thus, despite some pumping stations may have a higher elevation, if the resistance at this point is significative, other pumping

stations at lower elevations will be preferred to supply a more percentage of water into the network.

4.5. Cases study

To apply the methodologies presented three networks will be optimised. In the first cases, only the strategic models of the networks have been considered. That is, each network only contains the main pipes and the demand allocation is provided. The third network analysed is based on a more complex network.

The first network will be used as a comparative case, and it will be optimised applying both the D-M and C-M (i.e. H-J and N-M). The second and third network will be optimised only by mean of the C-M. Besides, the first two networks will be analysed from two points of view, considering NPDD and PDD. For doing that, an emitter exponent of 0.5 and an emitter coefficient of 0.8 will be assigned to all the consumption nodes. The networks considered in each case are:

a) TF network
b) CT network
c) COPLACA network

The starting parameters for H-J and N-M algorithms are presented in the next table.

Table 23. Parameters of the optimisation algorithms used for TF network

Notation	Description of parameters	Value
	Hooke and Jeeves	
E	Stop control value	0.001
D	Step length	0.1
	Nelder and Mead	
ρ	Reflection coefficient	1
χ	Expansion coefficient	2
γ_c	Contraction coefficient	0.5
σ	Shrink coefficient	0.5
E	Stop control value	1E-10

4.5.1. TF network

This system corresponds to one city of Spain that has 30,000 inhabitants and has been already introduced in example 3 (Figure 33). Next, two cases have been analysed:

- **Case 1:** When there are two pumping stations (PS1 and PS2).
- **Case 2:** When there are four pumping stations (PS1, PS2, PS3 and PS4).

The demand curve will be given for a minimum demand multiplier of 0.05, and a maximum demand multiplier of 2 with an increment of 0.05 for each stage of analysis. The number of combinations of the D-M will be performed for a $\Delta x = 5\%$. The minimum pressure required is $PH_{min} = 45\ m$.

4.5.1.1. Case 1: Two pumping stations PS1 and PS2

The network with two pumping stations and NPDD is shown in Figure 65. The information of the network has been presented already in Table 10 and Table 11.

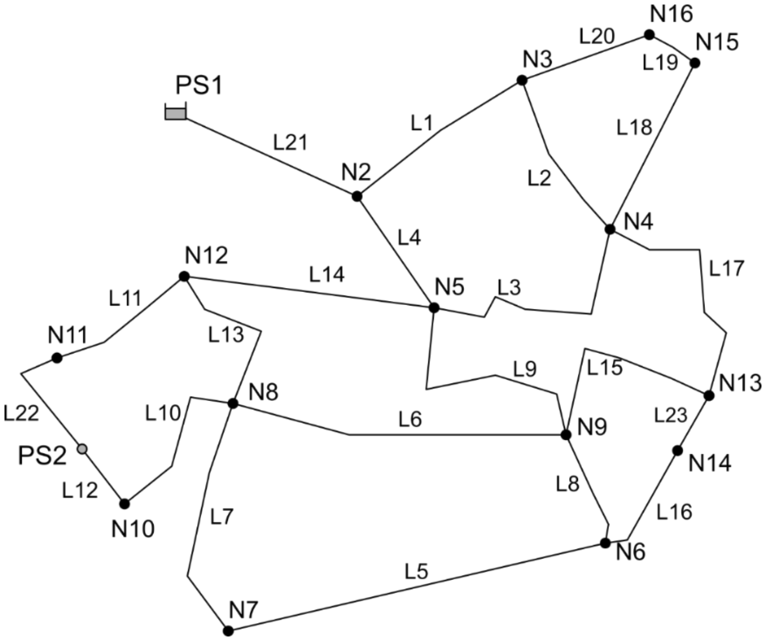

Figure 65. TF network with two pumping stations PS1 and PS2

Through the use of the D-M, the energy lines that define the optimal distribution for each demand are obtained. Energy lines are built by computing:

a) the required energy at pumping stations for each flow distribution and for each demand of the network, and
b) the flow provided by the pumping station (in percentage) regarding the network demand in a specific period.

Thus, for a specific network demand, there will be as many energy values as possible flow distributions. Then, all energy values are divide by the minimum value of all of them getting the total unitary energy of each combination (i.e. flow distribution). In that sense, the minimum energy will have a value of one. One energy line is built by drawing all the unitary energy values obtained for one network demand and the supplied flow (in percentage) by the pumping station according to the different flow distributions.

All the energy lines computed are shown in Figure 66 for the case of pumping station PS1 and Figure 67 for pumping station PS2. The x-axis shows the inflow of the pumping station (in percentage) regarding the total flow demand and y-axis shows the total unitary energy.

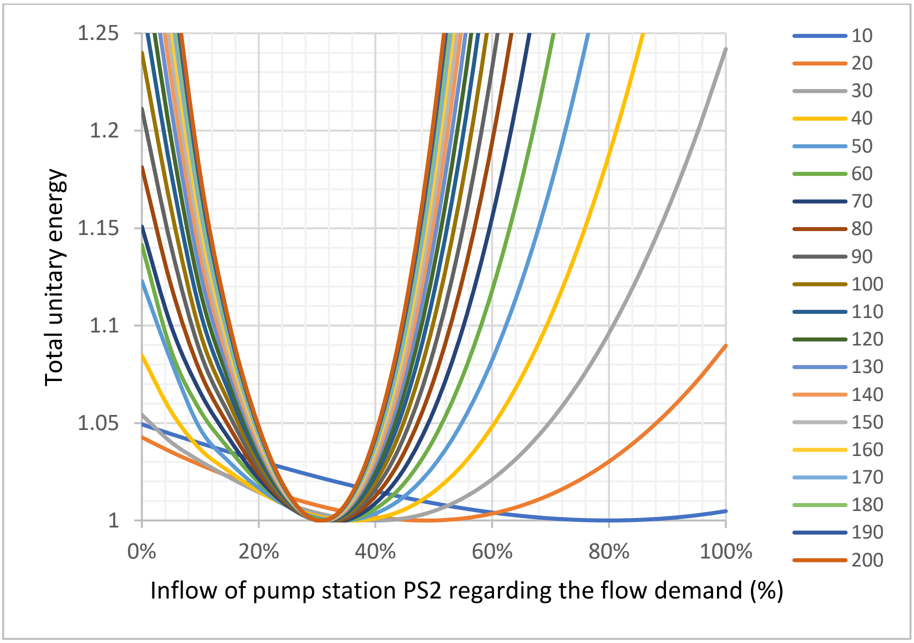

Figure 66. Inflow flow rate percentage of pumping station PS2 vs total unitary energy

It can be observed that for low values of the demand a higher percentage of the flow distribution is assumed by pumping station PS2 (Figure 67). For instance, when the demand is 10 l/s the percentage assumed is 80%. As demand increases, the percentage is reduced to 30%. Obviously, the rest of the demand is supplied by the pumping station PS1.

As the energy curves belongs to a pumping system with two pumping stations working at the same time, the energy curves of pumping station PS2 are a reflect of the curves of pumping station PS1. This because, the pumping stations complement each other. It is worth to highlight that the optimal flow distribution (i.e. distribution with the lowest energy requirements) is given when the value of 1 is reached in the y-axis. However, each energy curve is formed by several combinations of flow distributions that meet a same network demand.

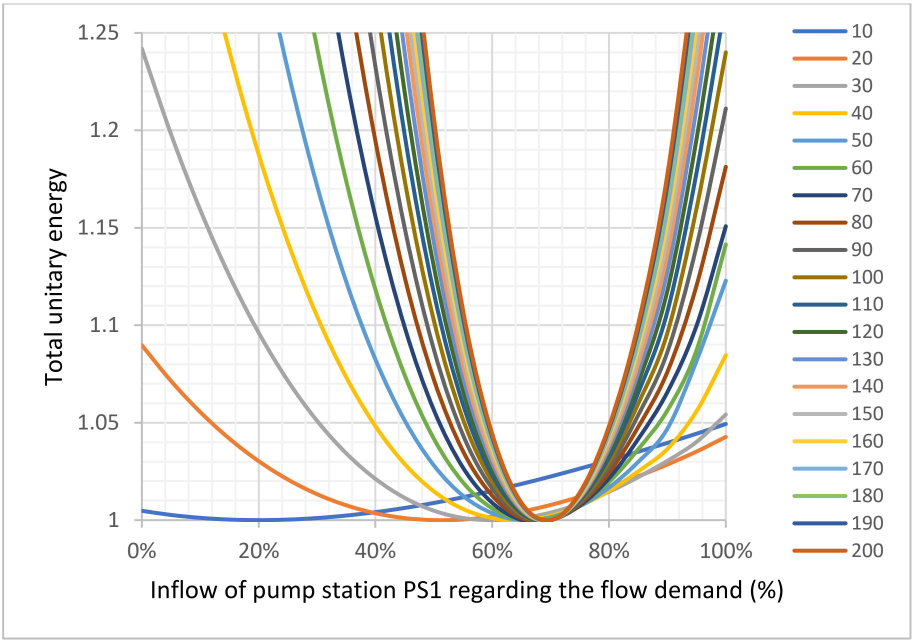

Figure 67. Inflow flow rate percentage of pumping station PS1 vs total unitary energy

Once the optimum distributions are known, the optimum distribution curves can be plotted (Figure 68). Despite the fact that points were obtained for a set for combinations with $\Delta x = 5\%$ and an increase in the demand factor of 0.05, it can be still noted that the distribution lines obtained have edges. In that sense, the next step will be the application of the C-M.

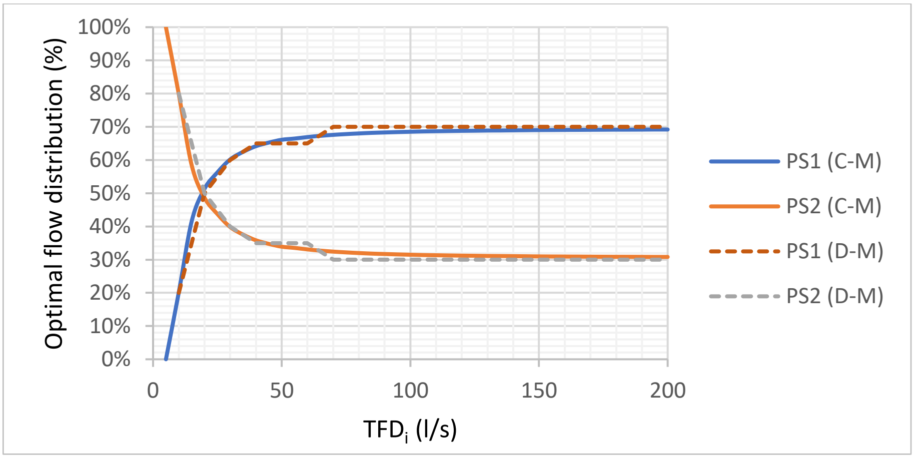

Figure 68. Optimal flow distribution of TF network when there are two pumping stations

When the C-M was applied and results of both algorithms (i.e. H-J and N-M) were compared, it was seen that they were the same. That is, if results of both algorithms are plotted in the same figure, there will be no difference. In that sense, the optimal solution has an additional guarantee. Therefore, only results of H-J will be presented (Figure 68). Actually, if N-M results are plotted with H-J results, there will be no difference.

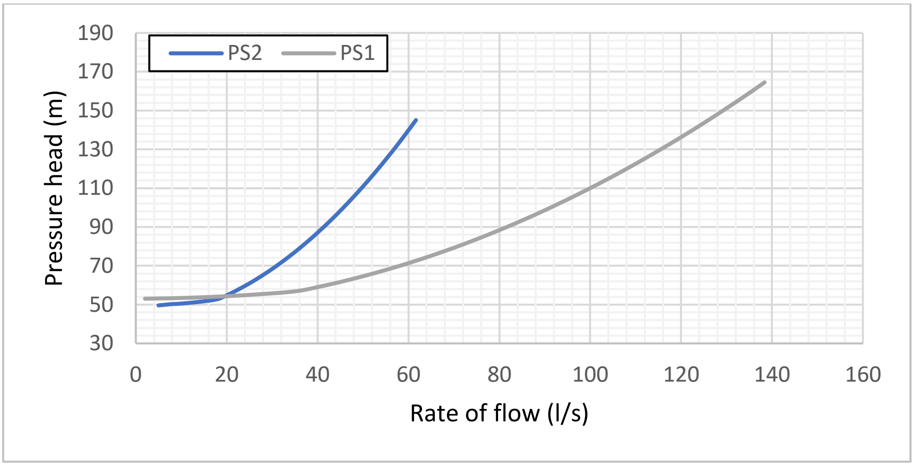

Figure 69. Optimal setpoint curves for PS1 and PS2

If results of the D-M and C-M are contrasted the curves of flow distribution are quite close to each other. However, when the distribution is considered as a continuous function more accurate is possible.

By taking the optimal flows of each pumping station and the pressure heads which correspond to those flows, SCs are obtained (Figure 69). It can be observed that at the beginning of the curves the gradient is different. As it has been pointed out before, this is a sign of a change of the location of the critical node. Besides, as pumping station PS1 is more efficient energetically, its curve is flatter and includes a bigger range of flows. Thus, the curves will be useful to sizing the pumping stations or for selecting a suitable method of operation.

Now, if the same network is analysed with PDD and the D-M is applied, it will be observed that the optimal flow distribution is much more defined, as energy curves show (Figure 70 and Figure 71). Probably, the cause is the increment of the demand due to the PDD. Thus, for high demands, the distribution is more stable as it began to be noted in Figure 68. Therefore, the optimal flow distribution between the two pumping stations will be constant over the whole range of demands. In that sense, pumping station PS1 will assume the 65% of the flow distribution and pumping station PS2 the remaining 35% (Figure 72).

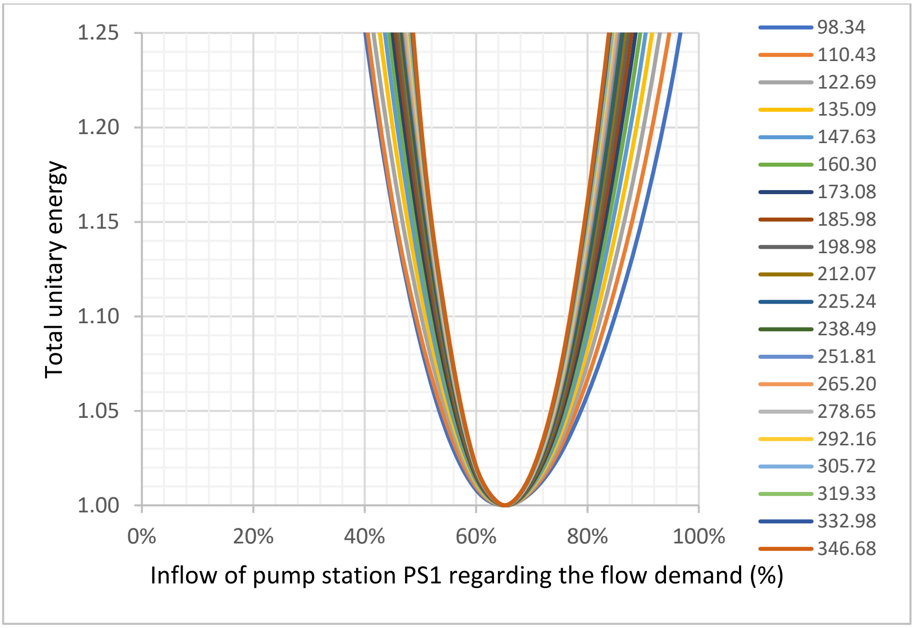

Figure 70. Inflow of PS1 vs total unitary energy when PDD is considered

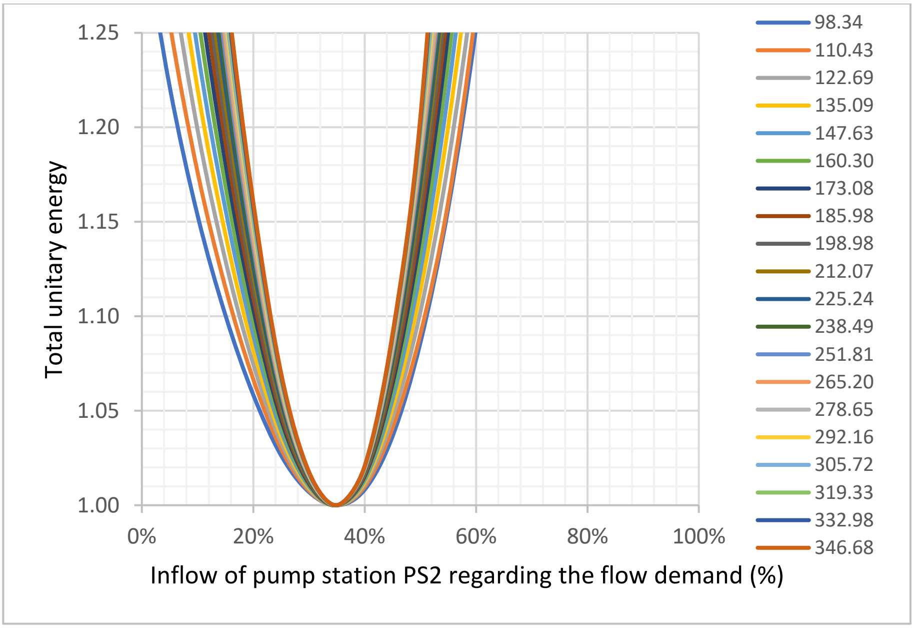

Figure 71. Inflow of PS2 vs total unitary energy when PDD is considered

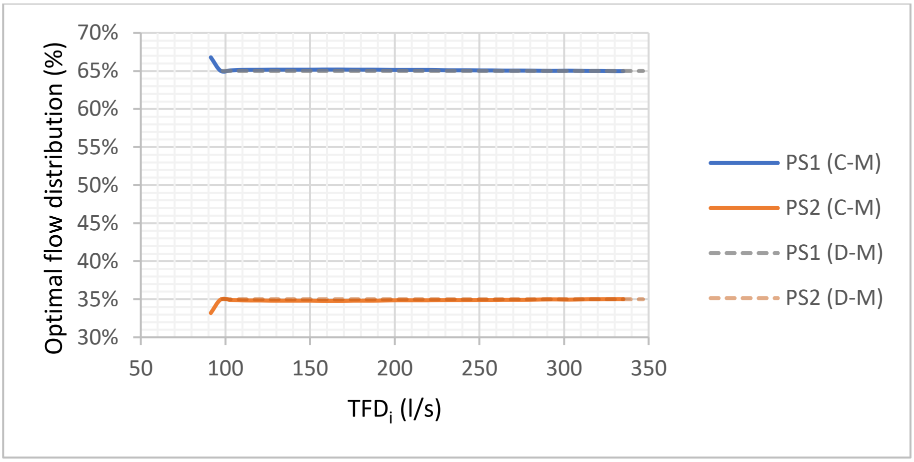

Figure 72. Optimal flow distribution between pumping stations PS1 and PS2 when PDD is considered

From the perspective of the C-M (Figure 72), the optimal distribution is quite similar, though the optimal distribution is not exactly 65% for pumping station PS1 since it presents little variations as well as happens with PS2. As it may be evident, the C-M offers more accurate results. The process ends by plotting the optimal SCs (Figure 73).

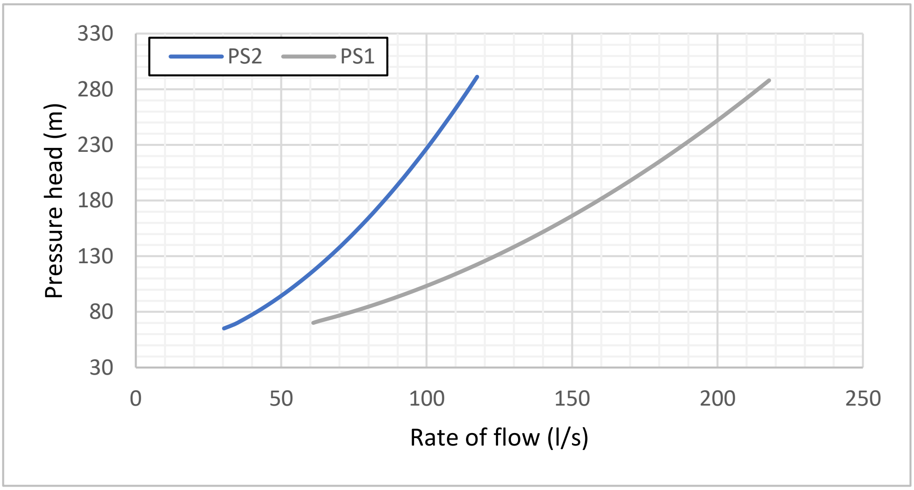

Figure 73. Optimal setpoint curves for pumping stations PS1 and PS2 when consumptions depend on pressure head

4.5.1.2. Case 2: Four pumping stations PS1, PS2, PS3 and PS4

In this case, two additional pumping stations have been added (PS3 and PS4). The head losses of the lines connecting the pumping stations to the network are negligible (Figure 74).

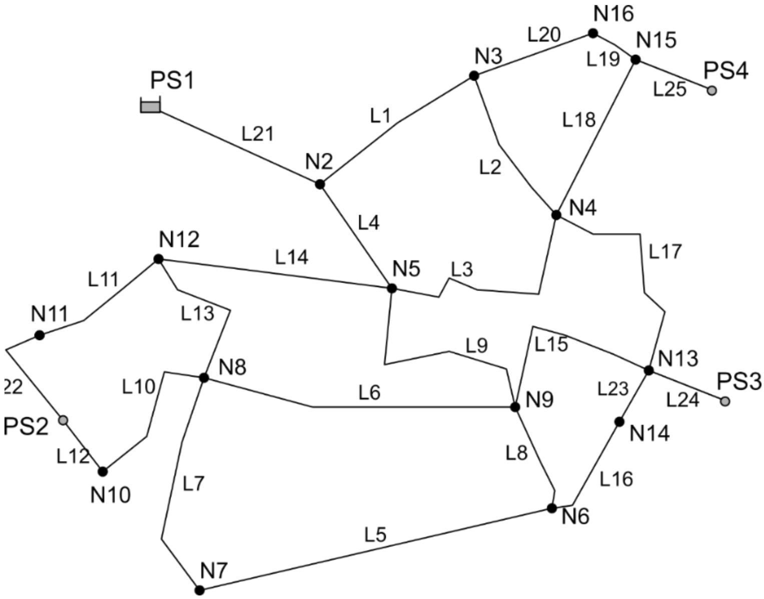

Figure 74. TF network with four pumping stations

When performing the optimisation either by the D-M or by the C-M (Figure 75) the order of importance according to the percentage of distribution is the following:

- PS1 (40%)
- PS2 (30%)
- PS3 (20%)
- PS4 (10%)

It can be observed that as the number of pumping stations increase, the D-M presents greater errors due to the rounding of the distribution percentage.

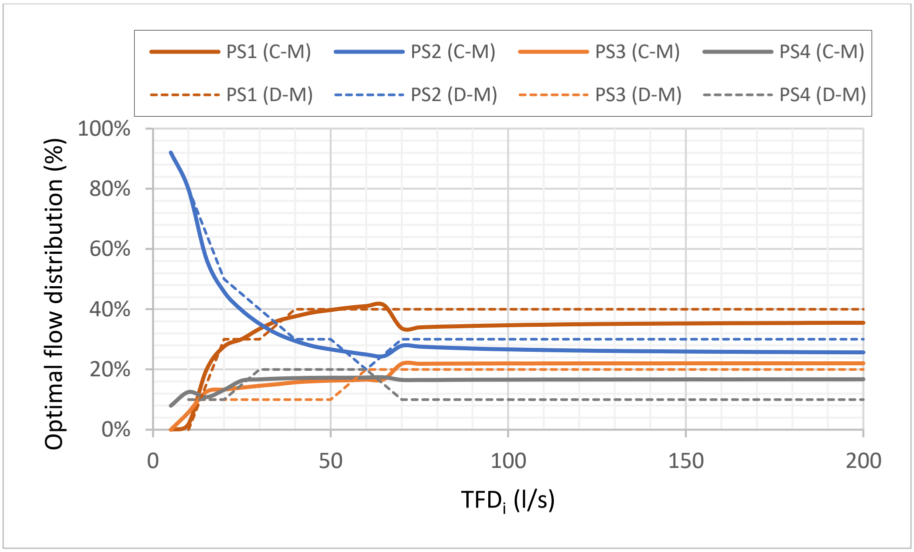

Figure 75. Optimal flow distribution for four pumping stations

So far, it had been observed that the optimum distribution was represented by curves having a uniform slope. However, when there are four pumping stations the flow distribution curves are more irregular. The reason is probably the variation of the location of the critical node when the demand changes. This last can be seen in the graphic of the optimal SCs (Figure 76), where there are sections with an almost flat slope which suddenly increase. In that sense, the SCs are quite similar to curves presented in Figure 35, Figure 36 and Figure 37.

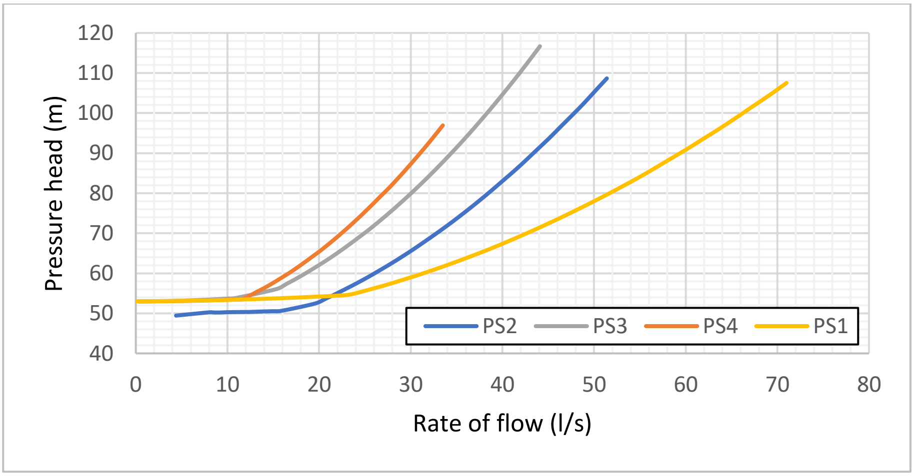

Figure 76. Optimal setpoint curves for pumping stations PS1, PS2, PS3, and PS4

When using a PDD model the optimal flow distribution among pumping stations is given by:

- PS1 = 35%
- PS2 = 25%
- PS3 = 25%
- PS4 = 15%

These distribution values are obtained from the C-M as Figure 77 shows. In this case, the optimal flow distribution is more defined, though there are some differences between the D-M because of the rounding process.

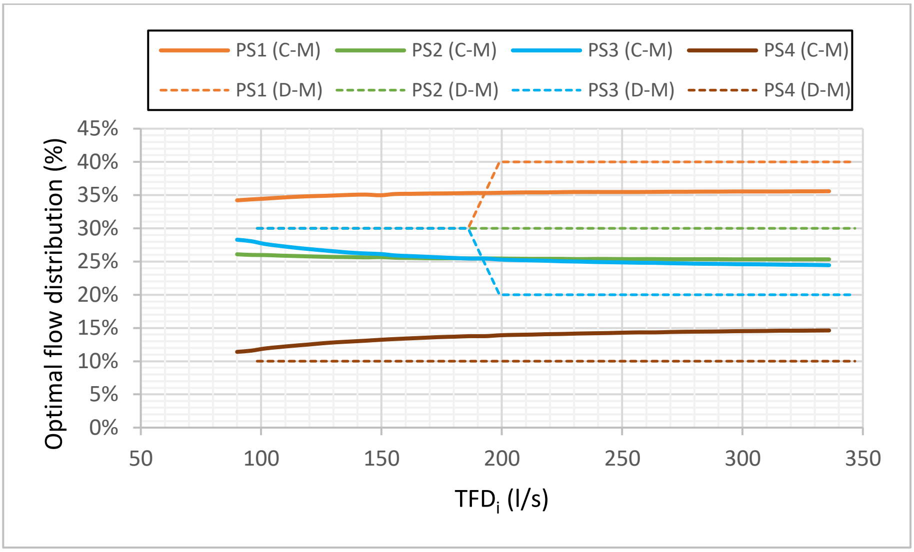

Figure 77. Optimal flow distribution for four pumping stations and PDD

Once the optimal SCs are plotted (Figure 78), it can be observed that according to the optimal flow distribution (Figure 77), pumping station PS4 with the lowest percentage of distribution has a curve with a higher slope (i.e. pumping station PS4). On the contrary, pumping station PS1 with a major flow distribution percentage has a SC with a lower slope. In that sense, it can be observed how SCs are defined by the optimisation process. This means, pumping stations with a lower energy consumption will have a flatter SC and a major flow range than pumping stations with major energy needs.

It may be interesting to mention that the optimal SCs also could be got by calculating the constant flow distribution with the lowest energy consumption requirements. In that context, the optimisation will require being made for the whole demands of the network at once instead of separately as it is done in the exposed methodology.

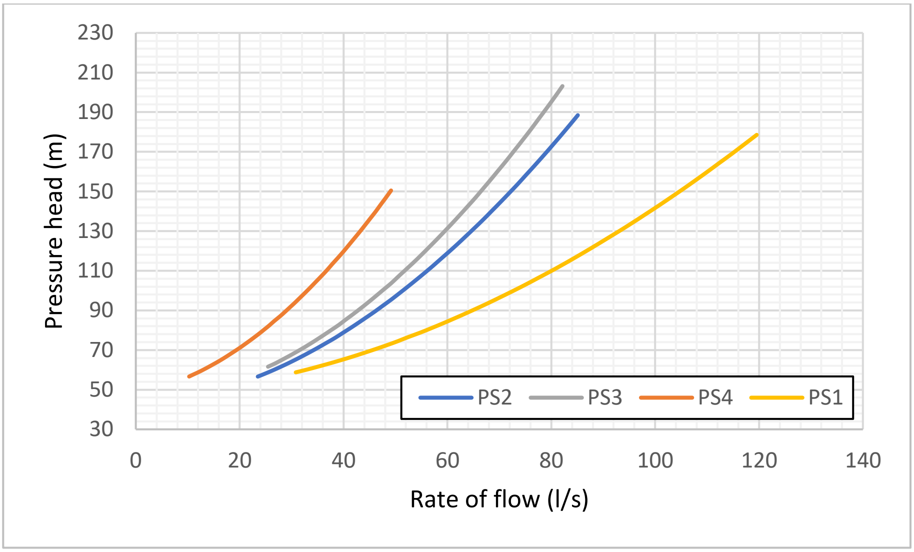

Figure 78. Optimal setpoint curves for pressure dependent consumptions and four pumping stations

4.5.2. Catinen Network

This network corresponds to a city of 50,000 inhabitants in Spain (Figure 79). It has 30 pipes, and 21 nodes, three of them are pumping stations (see, F1, F2 and F3) associated with water sources and the remainder are demand nodes. The minimum pressure required is PH_{min} = 45 m. The average flow rate demanded is 154.20 l/s. Despite pumping stations are in the extremes of the network, their HGL elevation at suction is the same, zero.

It is assumed that the network demand goes from 0.05 to 2 times the average flow demand in increments of 0.05. Thus, the SCs will be formed for a total of 40 points. This is, the network has 40 demands.

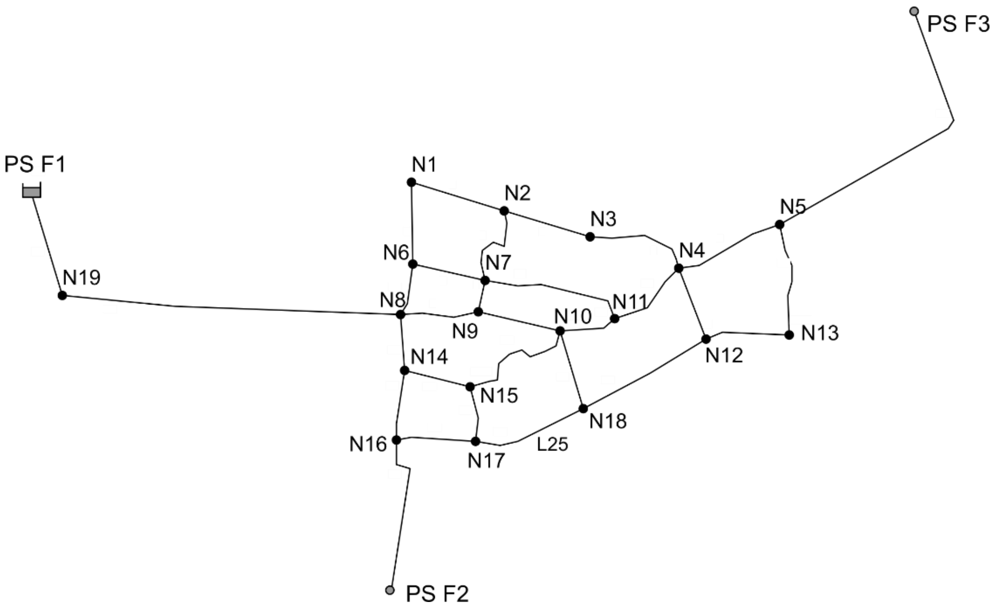

Figure 79. Catinen network

The information about nodes and pipelines is described in Table 24 and Table 25. Most of the pipes have a roughness of 0.03 and a few a roughness of 0.10

Table 24. Junctions of Catinen network

ID	Elev. (m)	Demand (l/s)	ID	Elev. (m)	Demand (l/s)
N1	9.0	11.9	**N12**	7.5	9.4
N2	7.0	7.4	**N13**	8.5	9.6
N3	5.0	10.3	**N14**	9.6	8.8
N4	7.5	4.6	**N15**	7.8	5.3
N5	10.0	17.5	**N16**	10	13.8
N6	9.6	5.1	**N17**	7.8	4.3
N7	8.0	4.9	**N18**	6.0	8.4
N8	9.9	11.0	**N19**	6.0	4.4
N9	7.8	3.7	**F3**	0.0	0.0
N10	6.0	7.5	**F2**	0.0	0.0
N11	5.3	6.3	**F1**	0.0	0.0

Table 25. Pipes of Catinen network

Node 1	Node 2	Length (m)	Diam. (mm)	Roug.	Node 1	Node 2	Length (m)	Diam. (mm)	Roug.
N1	**N6**	253.26	199.2	0.03	**N12**	**N13**	268.10	148.4	0.03
N2	**N1**	301.88	148.4	0.03	**N12**	**N4**	191.92	199.2	0.03
N2	**N3**	260.79	199.2	0.03	**N5**	**N13**	391.53	123.0	0.03
N3	**N4**	345.08	123.0	0.03	**N4**	**N11**	268.24	148.4	0.03
N4	**N5**	342.25	148.4	0.03	**N8**	**N14**	169.26	250.0	0.10
N6	**N7**	211.13	148.4	0.03	**N14**	**N15**	239.94	250.0	0.10
N7	**N2**	301.81	199.2	0.03	**N15**	**N10**	384.76	123.0	0.03
N7	**N9**	113.47	199.2	0.03	**N15**	**N17**	165.81	148.4	0.03
N9	**N8**	215.97	250.0	0.10	**N17**	**N16**	261.97	199.2	0.03
N8	**N6**	146.87	199.2	0.03	**N17**	**N18**	354.56	148.4	0.03
N7	**N11**	459.60	199.2	0.03	**N19**	**N8**	1047.55	498.0	0.03
N11	**N10**	142.14	150.0	0.10	**N14**	**N16**	204.87	199.2	0.03
N10	**N9**	306.66	199.2	0.03	**F1**	**N19**	150.00	498.0	0.10
N10	**N18**	222.95	148.4	0.03	**N5**	**F3**	2000.00	199.2	0.03
N18	**N12**	438.65	148.4	0.03	**N16**	**F2**	1300.00	199.2	0.03

Since both D-M and C-M have already been compared in the optimisation of TF network, the optimisation of the Catinen network will be done only through the use of the C-M. The parameters applied to the optimisation algorithms are the same presented in Table 23.The analysis will be performed first for NPDD and then for PDD. For PDD an emitter exponent of 0.5 and an emitter coefficient equal to 0.8 for each demand node will be used.

The assumption for the starting point needed for the application of H-J algorithm is that at the beginning the whole demand is supplied by F1 pumping station. Once the optimisation has been carried out (Figure 80), the distribution remains relatively constant along the increase of the demand. In that context the analysed network is balanced. It can be seen that pumping station F1 has the lower energy requirements since it is advisable that a percentage of almost 74% of the demand is assigned to it. The second place is for pumping station F3 with around 18% of the flow distribution. Finally, F2 is assigned a flow distribution of approximately 8%. At this point, the information obtained can be useful to find out where is more suitable a higher investment in case that capacity of sources must be improved. The optimal SCs are presented in Figure 81.

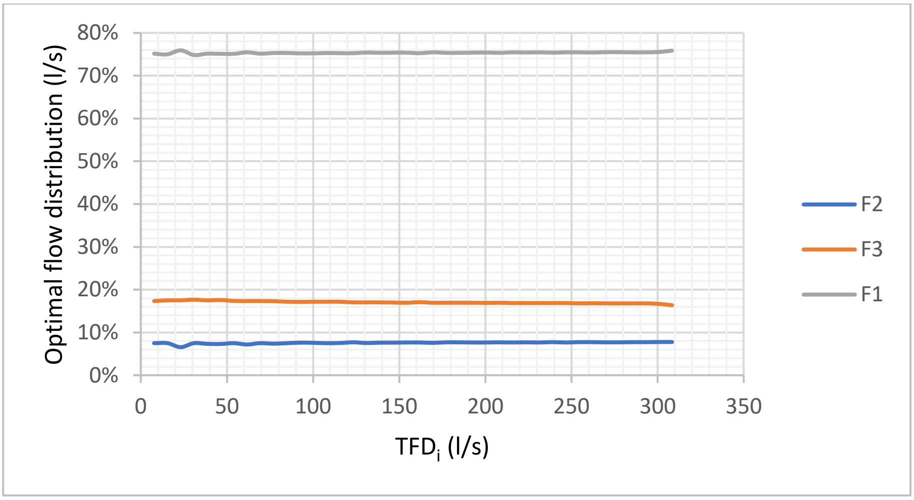

Figure 80. Optimal flow distribution of Catinen network

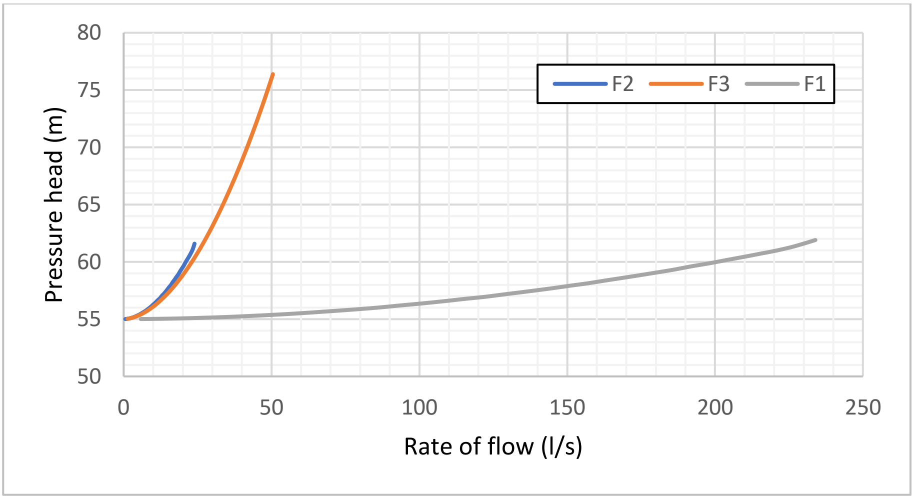

Figure 81. Optimal setpoint curves for consumptions with NPDD

In the case of PDD the average of the optimal flow distribution is the following:

- F1 = 76%
- F2 = 8%
- F3 = 16%

In that sense, the optimal flow distributions seem slightly different but not too much in comparison with NPDD model (Figure 82). However, the difference is more apparent when optimal SCs are plotted since the range of flow rates to be supplied by each pumping station is entirely different (Figure 83). Besides, as it already happened in other examples the change in slope of the SCs reflects the variation of the critical node as the demand increases.

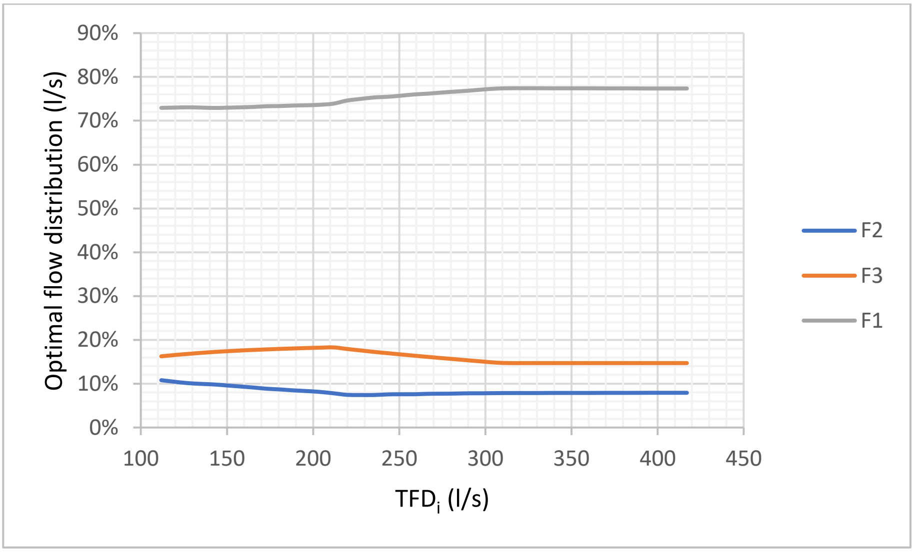

Figure 82. Optimal flow distribution for Catinen network with PDD

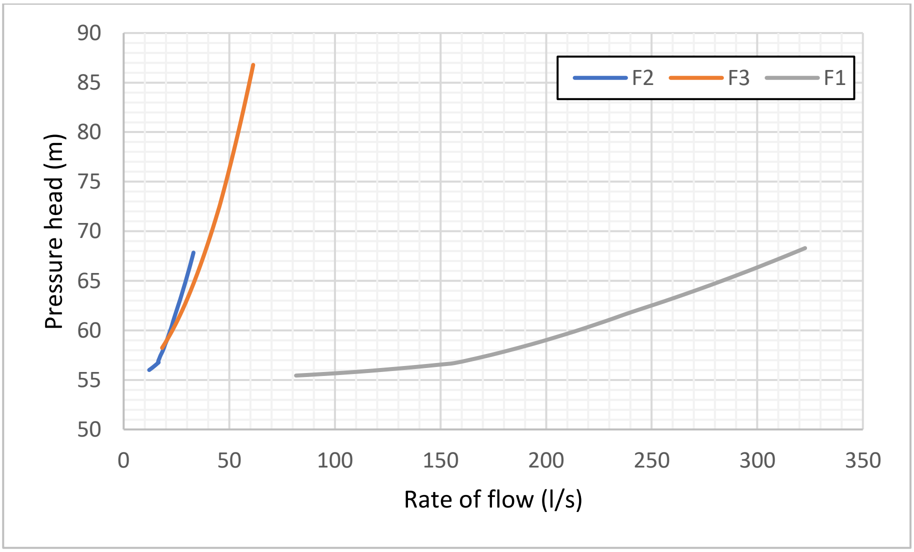

Figure 83. Optimal SCs for Catinen network with PDD

4.5.3. COPLACA network

The model was built for a real city of Spain with a population of 25,000 inhabitants (Figure 84). It has a seasonally variable demand, which is particularly difficult to satisfy in the summer. The residential area does not receive enough water due to the increased water demand. Therefore, the municipality is considering additional water resources, which would involve reactivation of some old and neglected wells. The distribution network consists of 1,032 nodes, 1,095 pipes (a total length of 133 km), and one reservoir. There are seven water sources; six of them are nodes that represent pumping wells: P05, P06, P07, P11, P12, and P13. Each well has a maximum extraction flow rate (Q_{max}) associated with it. Reservoir P10 represents a river source, which supplies water through a pumping station. Consumption is considered pressure dependent. The minimum required pressure is 20 m.

Figure 84.COPLACA network

Also, a minimum flow rate (Q_{min} = *0.5 l/s*) for each water source has been fixed to avoid solutions with unrealistically low flow rates.

Table 26. Maximum rate of flow allowed per pumping station

ID	Q_{max} (l/s)
P05	9
P06	3
P07	7
P11	17
P12	15
P13	15
P10	80

The analysis will be performed for a minimum demand multiplier of 0.05 and a maximum demand multiplier of 2 with increments of 0.05. The parameters for the C-M are the same as the previous cases.

Once the flow distribution optimisation has been carried out Figure 85 is obtained. The x-axis shows the total flow demand at time *i* (i.e. network demand), and the y-axis the percentage of flow that is supplied by a certain pumping station in order to satisfy the network demand.

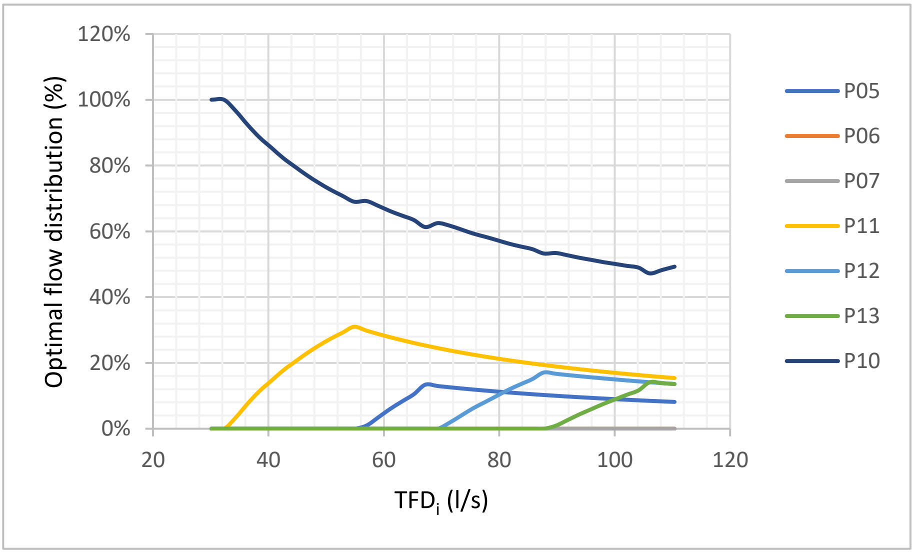

Figure 85. Optimal flow distribution of COPLACA network in percentage

In the figure can be noted that, when the demand of the network is, per example, 40 l/s, the pumping station P10 supplies the 85% of the network demand and the remaining 15% is supplied by the pumping station P11. In that context, values of 0% mean that pumping stations are not working and are not needed to satisfy the demand. As the demand of the network increases, the number of pumping stations in operation also increases. The flow distribution associated with each pumping station increase until reach a maximum. After that, it seems that the slope of the curves decreases. In that context, it has to be kept in mind that while more pumping stations are working less percentage of the flow distribution is assumed by each one of them. Despite there being seven pumping stations available, only five of them participate in the optimal flow distribution (P11, P05, P12, P13 and P10). Thus, the analysis shows that it is not required to use all the available wells to satisfy the demand. As the pumping station associated with the river has a higher elevation, it assumes a higher percentage of the flow distribution, but the second place is for the pumping station P11 which is on the other side of the network. To show in a better way the incorporation of new pumping stations when their maximum flow rate is reached Figure 86 has been plotted. Besides, it can be appreciated little variations in the flow rate supplied by pumping station P10 as result of the limitations of the flow rate of the other pumping stations.

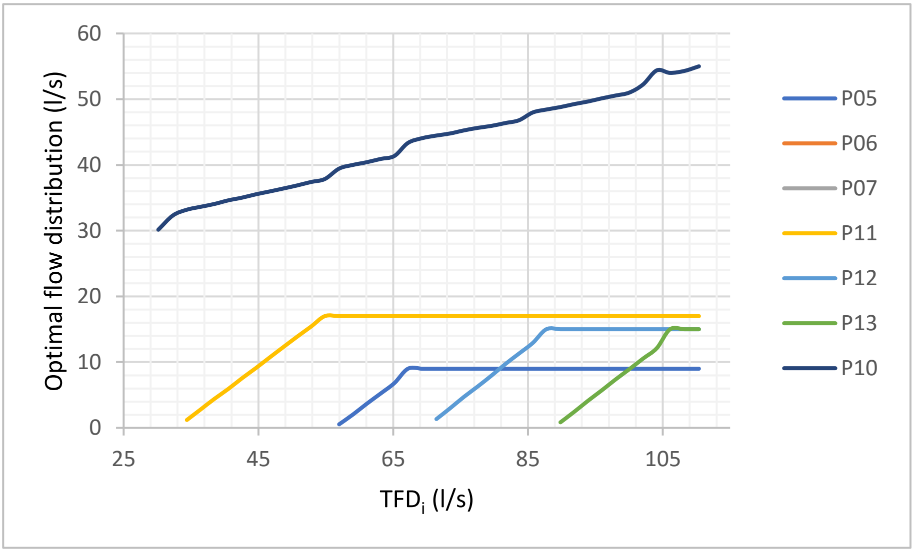

Figure 86. Optimal flow distribution of COPLACA network in litres per second

The flow supplied by each pumping station regarding the network demand in a specific moment is presented in Figure 87. In the figure, as the network demand increase, the interaction among the pumping stations can be observed. Thus, each time that a pumping station reaches its maximum flow rate a new one is summed to the flow distribution.

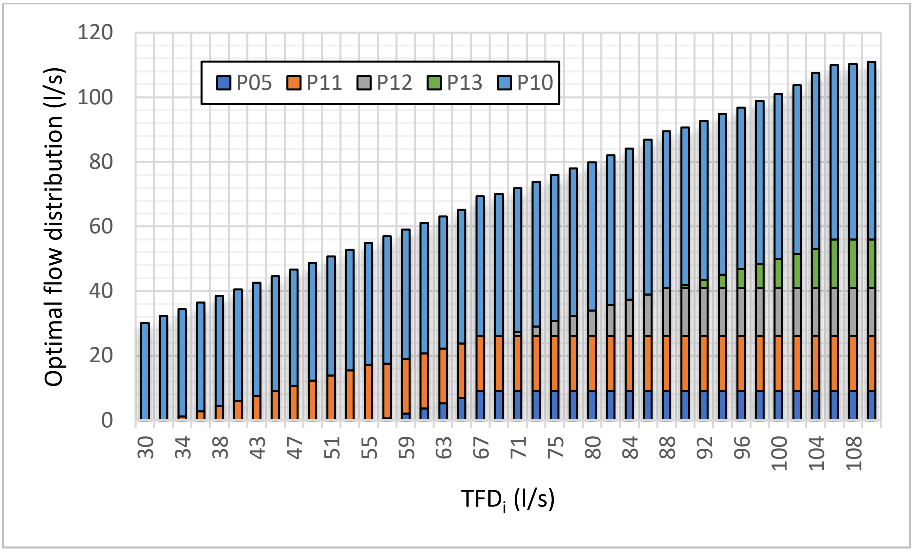

Figure 87. Supplied flow by the pumping stations according to the network demand

The optimal SCs are shown in Figure 88. In the case of this specific network, it results interesting to observe that the curves are quite flat which suggest that there are no significant changes in the elevation of the demand nodes. The real shape of the SCs can be seen if the curves are drawn separately and its scale is adjusted.

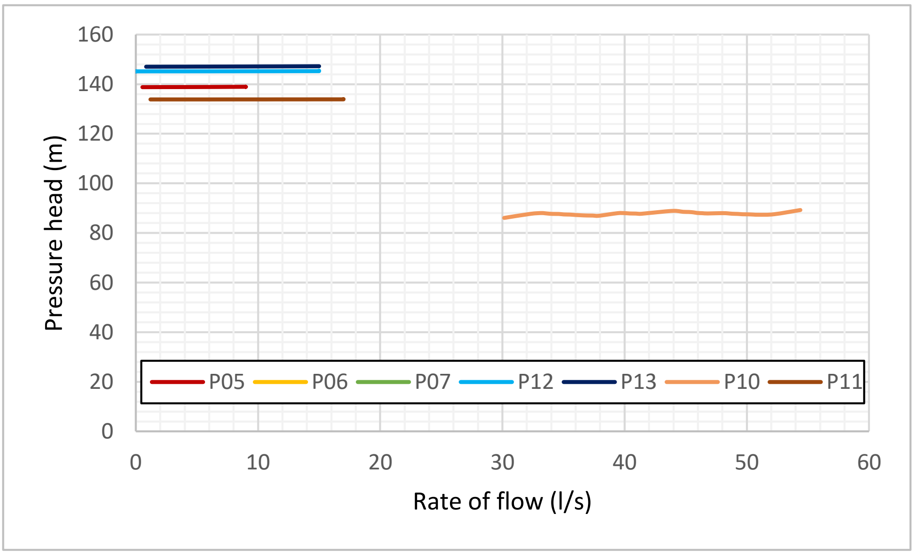

Figure 88. Optimal SCs of COPLACA network

The optimisation of the distribution achieved does not only show the most economical energy situation since it has the added value of indicating the scheduled operation of the sources of supply. Additionally, the optimisation shows those pumping stations that are not suitable for the flow distribution or that are not necessary.

Concluding this chapter, it can be said that the pumping energy optimisation carried out through the SC concept allows knowing the optimal flow distribution among water supply sources. This can be done by two methods: D-M and C-M. The difference between the methods lies in the way of assessing the different possible combinations of flow distributions but at the end, results are quite similar. On the other side, independently of the method, the optimal distribution obeys the condition of hold the minimum pressure at a reference node (i.e. the critical node) of the network. This condition is kept always even when the flow distribution change. Thus, starting from the optimal flow distribution, it is possible to reach a unique SC for each pumping station. The SCs are built by joining the points of pressure heads and flow rates of the pumping stations got from the optimal distribution. In the cases study, it can be observed that optimal flow distribution allows identifying the order of importance of each pumping station as well as their interaction with the other supply sources. Besides, the SC application results of very importance for the pumping stations sizing and generation of politics of operation. In this way, the process carried out is useful not only for the energy optimisation but the better management of the water networks.

Chapter 5

Cost optimisation without storage capacity

In the previous chapter, pumping energy optimisation was presented. The process was developed by searching the optimal flow distribution among pumping stations. Then, starting from the optimal distribution, the optimal setpoint curves (SCs) are built. These curves are the system head curves (SHCs) that pumping stations must follow to keep the minimum nodal pressure of the network while the demand is satisfied. In that sense, SCs point out the values of pressure head and flow rate required in the supply sources to keep the minimum energy consumption (i.e. in terms of pressure head) of the network. So far, costs have not been considered within the optimisation process. Though, energy optimisation already leads to economic savings due to the estimation of excess energy associated with pump performance curves. Since pump head curves are concave for a decreasing flow the pumping head provided is greater than the really needed. Therefore, the extra power supplied by pumps resulting in lack of efficiency. However, this is insufficient from the perspective of the optimisation of the operating costs, because due to the complexity of electricity rates, energy savings do not always coincide with cost reductions. Therefore, starting from the energy optimisation, a new approach based on the optimisation of the pumping operating costs is formulated. The aim is to find the least cost SCs which will be the guide to achieve the minimum pumping operating costs. This technique could be combined with methods for the optimal sizing and selection of pumps, though this last is out of the scope of this research. In fact, their consideration will lead to a more complex problem that involves the determination of the optimal number of pumps, the kind of pumps (i.e. variable or fixed speed pumps), their optimal operation method (i.e. valves control use, by-pass applications, etc), equipment required to program the pumps operation (flow meters, manometers, pressure sensors, etc.)

among other aspects. In that sense, capital costs are not included since their consideration has implications which are out of the limits of this study.

Before going on with the formulation of the objective function (OF), it is essential to make a fast review of the problem features that will be addressed. The approach uses the concept of a SC, where the most critical node in the network is identified, and all of the pumping stations are represented as nodes [7]–[9]. The critical node is used as a reference point to optimise pressure heads at pumping stations and satisfy the pressure requirements in the network while keeping the energy consumption at the minimum. The critical node can change depending on both the topography of the network and the changing demand in the system. Therefore, it has to be found for each instant (step). By minimising the pressure in the network, the leakage is also reduced, and associated additional benefits are achieved. A concept similar to the SC is the resistance curve (RC), proposed by Walski [55]. Walski presents the RCs depending on whether there is a tank in the network or not. In the case of closed systems, it is necessary to know the resistance of all elements in order to determine the RC of the network. That is the origin of the methodology based on the SC: how to obtain these curves when we do not know the resistance of the consumers?. For this reason, it is necessary to consider the minimum pressure at the critical node and from that point determine the required head in each source.

SC might be defined as a theoretical curve that points out the minimum energy (regarding pressure head) required on source points (storage, pumping station) to meet the minimum pressure needed for each demand in the network (see, section 2.4.2.). That is, it is a representation of the pressure head versus flow at a given point in the system. In many cases, the RC ("system head curve" defined by Walski et al. [55]) is confused with the SC. Nevertheless, they are not the same as it was already demonstrated in previous sections (see, section 2.4.3.).

The SC concept does not require to model a pump as a hydraulic machine with its pump characteristics (e.g., pump performance curve, efficiency curve, and power curve). It uses instead a node that represents a conceptual (hypothetical) pump, where for a given flow rate to be supplied at that node, the model determines the pressure head needed to satisfy the required flow rate. The values of both flow rate and pressure head for the conceptual pump are limited only by the required demand and the minimum pressure in the network.

As the SC deals with hypothetical pumps with no limitations on flow rate, it is not possible to associate an efficiency curve with them. However, as that is important to determine costs associated with flow rate and pressure head, the assumption taken here is that a constant efficiency value can be applied.

For this reason, it is proposed in this work the determination of the optimal distribution of flows among the different sources of a water supply system. For each flow distribution, it is necessary to assess the energy cost that would have the pumping station operation installed in each source. In the event that the pumping station exists, and it is not desired to modify its characteristics, this OPEX (operational expenditure) will be evaluated from the head and efficiency curves of pumps. If the pumping station can be designed, the energy consumed is obtained from the flow rate, the head value on the SC and an estimation of the minimum efficiency with which the pumping station would work. In the latter case, the variation of the efficiency of the flow has not been taken into account, since the pumps have not yet been selected. However, the definition of a minimum efficiency sets the criteria for pumps selection.

Obviously, the flow rate provided by the sources must meet demand requirements in the network. In the case of a system with several sources, the flow supplied by each source can take many different values. Every flow combination defines a specific SC for every source, which will also maintain the required minimum pressure. Hence, there are as many SCs as source discharge combinations that will meet overall demand, but there is only one that carries the minimum energy consumption. That is the optimal SC. Thus, the cornerstone of this work is to find the optimal SC that leads to the minimum energy cost through the formulation of a cost objective function and the use of an optimisation algorithm.

The underlying assumptions of the methodology are as follows:

a) Multiple water sources are available to supply water for consumption in the network.
b) Each of the sources has its pump(s) station, but the selection and sizing of the pumps is out of the scope of this research.
c) Storage is not available on the network, and only snapshot hydraulic analysis is required to describe hydraulic behaviour of the system.
d) A SC for each pumping station is obtained.
e) No emergency events (fires, pipe breaks and unusual demands) are considered.
f) The demand for nodes and demand patterns are known. This means that the flow rate and pressure head for each supply source are determined such that it satisfies both the demand in the network and the minimum pressure required.

5.1. Objective function

The least-cost solution is determined by optimising the OF that is formulated as the sum of the two cost terms. The first one represents the pump energy cost and the second is the cost of water treatment. The methodology is generalised for any duration and any time step. However, this work has been developed for a 24 h time horizon with 1 h intervals. Therefore, 24 results are obtained at the end of the simulation period.

The first term in the OF is obtained by multiplying the power consumption at the pumping station with the tariff unit charge and pumping time. In this case, the tariff function is represented as an average value per hour. Hence, it depends only on the energy consumed over the day.

$$PEC_i = \sum_{j=1}^{Nps} \frac{\gamma \cdot Q_{ij} \cdot PH_{ij}}{\eta_{ij}} \cdot ET_{ij} \cdot t_i \qquad \textbf{(66)}$$

*W*here,

PEC_i is the pumping energy cost in period *i*. This is, the sum of the power consumption cost for each pumping station *j* at hour *i*,
Nps is the number of pumping stations,
γ is the specific weight of water,
Q_{ij} is the flow rate for each pumping station *j* at hour *i*,
PH_{ij} is the pressure head needed at each pumping station *j* at hour *i*,
η_{ij} is the minimum efficiency estimated for each pumping station *j* depending if it is desirable to incorporate partially the characteristics of pre-existent pumps or whether it is a new pumping system,
ET_{ij} is the energy tariff at hour *i* at the pumping station *j*, and
t is the pumping time at hour *i*.

The flow rate and the pressure head identify points on the SC for each pumping station. It is worth to highlight that the flow rate of each pumping station is given by Equation (50). Thus, the variable of decision of the OF is still being $X_{i,j}$. It is important to note that as the proposed methodology involves simulation of the behaviour of unknown pumps, the efficiency used should be understood as an estimation of the minimum efficiency of the pumping station once all the pumps have been selected.

If the efficiency value is taken as the maximum efficiency, the optimisation leads to sub-optimal solutions since pumping stations cannot work at the maximum efficiency all the time. However, it could result interesting use the maximum efficiency to find out the maximum possible savings if the pumps always were operating in conditions of maximum efficiency. This could be interesting from the point of view of the optimal selection of pumping systems and the regulation of their operation. The reason is that once the maximum possible savings are known, it is also possible to visualize the scope of the efforts that can be made to optimize costs.

The second term of the OF, i.e., the cost of the treated water, is calculated as the product of the pumped flow rate and the cost of a cubic meter of treated water. Those costs are directly related to the volume of water produced.

$$TWC_i = \sum_{j=1}^{Nps} \left(TC_{ij} \cdot Q_{ij} \cdot t_i \right) \quad \textbf{(67)}$$

Where,

TWC_i is the sum of the treated water cost for each supply source *j* at hour *i*,

TC_{ij} is the unit treatment cost for each water source *j* at time *i*. This value could also depend on aspects such as disinfection chemicals, maintenance, energy for the plant devices, and others.

The OF can be expressed as follow:

$$\sum_{i=1}^{24} Min\, f(x)_i = \sum_{i=1}^{24} (\, PEC_i + TWC_i) \quad \textbf{(68)}$$

There are two types of constraints on the problem. The first are those related to the hydraulics of the system:

a) flow and energy conservation constraints,
b) pressure constraints, and
c) no negativity constraints for some variables.

The second type of restrictions is introduced to avoid infeasible solutions being evaluated:

a) The addition of each flow rate supplied to the network must be equal to the total flow rate (TFD_i) required at time *i*:

$$\sum_{j=1}^{Nps} Q_{ij} = TFD_i \quad \textbf{(69)}$$

b) The total flow rate supplied by a water source cannot be higher than the total flow rate required and must be greater than zero:

$$0 \leq Q_{ij} \leq TFD_i \quad \textbf{(70)}$$

c) The pressure head at the critical node (PHc_i) in the network must be equal to the minimum pressure head required (PH_{min}):

$$PHc_i = PH_{min} \quad \textbf{(71)}$$

5.2. Energy tariffs

There are different kinds of energy tariffs: standard tariffs, fixed energy tariffs, dual fuel tariffs, online, energy tariffs, pre-payment tariffs, ‘Green’ energy tariffs and others. The variety of all of them depends on the country regulations, market, type of consumer among others. For instance, in some countries, the prices of the electricity are calculated in real time and can go up or down depending on the energy market conditions. Thus, there is a different price each hour. In other cases, energy tariffs are calculated according to the average price of the energy over the billing period. Another way of billing is by means of setting different prices according to different periods of the day. These periods are known as: peak hours, off-peak hours, etc. Besides the hours of the periods can change depending on the season (i.e. summer and winter). In this context, there is a huge number of methods to set the price of the energy. However, in general terms, the energy price is computed by two terms, one is variable and the other is fixed. The variable term has been already introduced and consist of the price of the energy consumed in a specific time. On the other hand, the fixed term involves the maximum power contracted. The maximum power has a direct relation with the number of the electric equipment that can be working at the same time. In the case of small consumers, when the maximum power is exceeded the energy supplied is cut off automatically by means of an energy controller and it has to be turned on again. However, in the case of bigger consumers, such controller does not exist. Thus, when the maximum power contracted is exceeded a penalty cost is billed. Further, in some case when the power consumed is lower than the contracted a penalty cost also is considered. In that sense, the final energy price will be given by the addition of the two terms.

As it can be supposed, this research is based on the energy cost optimisation of the big consumers (i.e. pumping stations). However, it has been mentioned several times throughout the document that the sizing of pumping stations is out of the scope of the research. In that sense, it is not possible to know neither the number, the type nor the power consumption of the pumps since they have not been selected yet. In consequence, the term of maximum power contracted has not been considered in this work. Of course, it must be included when sizing of pumping stations will be done.

5.3. Cases study

To apply the operating cost optimisation of pumping it will be used two of the networks that have been already presented: TF network and COPLACA network.

5.3.1. TF network

The information of the system has been shown previously. Therefore, only the additional information to perform the analysis will be added. The optimisation will be developed for pressure-driven demands (PDDs). Thus, the emitter exponent used is 0.5, and the emitter coefficient for each node is 0.8. Only three pumping stations are considered PS1, PS2 and PS3 (Figure 33).

The network demand variations are given by Figure 89 and the energy tariffs have been discretised into four periods, input data is shown in Table 27. The prices correspond to the energy tariff term in the expression of the power consumption (Equation 66). The maximum power has not been considered in this work as has been explained (see, section 5.2.). Both, the minimum efficiency and the cost of the water treatment are given for each source and they are assumed constant over time (Table 28). However, the methodology could be applied equally if the costs or efficiency vary over time or with the flow. The minimal required pressure is 45 m.

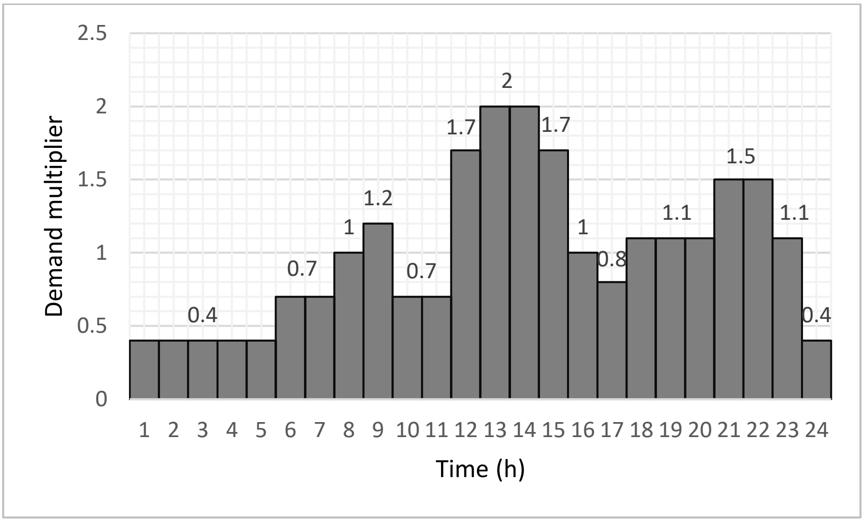

Figure 89. Demand variation of TF network

Table 27. Energy tariffs regarding the hour and the water source of TF network

Time (h)	PS1 (€/kWh)	PS2 (€/kWh)	PS3 (€/kWh)
1-8	0.094	0.092	0.090
9-18	0.133	0.131	0.129
19-22	0.166	0.164	0.162
23-24	0.133	0.131	0.129

*Electric tariffs are variations of ENDESA (2017)

Table 28. Efficiencies and treatment costs

Pumping station	η (%)	TC (€/m3)
PS1	60	0.30
PS2	75	0.25
PS3	65	0.20

Once the optimisation is carried out the optimal flow distribution over the period of simulation is plotted in Figure 90. Apparently, the distribution between pumping stations PS2 and PS3 is the same over the whole horizon of simulation. In the case of PS1, it seems like the pumping station is less important for low values of the demand and its contribution becomes more critical when the demand increase.

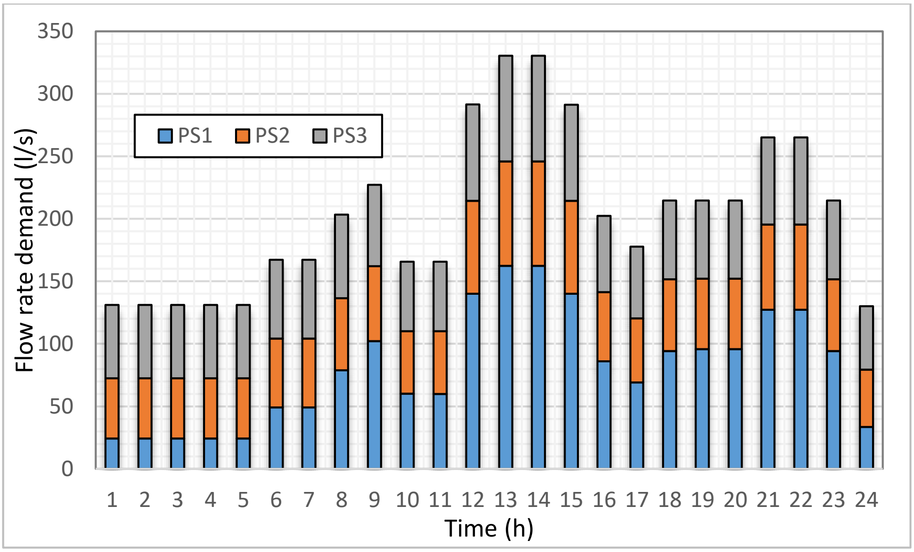

Figure 90. Optimal flow distribution over the time in l/s

Although source PS2 is at a higher elevation than PS1, the optimisation results show that PS1 is preferred over PS2, i.e., the minimum energy curve is associated with PS1. In other words, it is beneficial for source PS1 to provide more water to the network than PS2 and PS3 in order to minimise the operating costs. Therefore, this demonstrates that the problem of finding the SCs is a complex one, which cannot be understood by just observing pressure heads at various sources but can only be solved by using an optimisation approach. The optimised solution shows a flow distribution among the sources (Figure 91), which has not been obvious before optimisation. For the first seven hours of operation, the source PS3 makes the largest contribution to the system flows. This is logical as that is the source with the lowest electricity tariff for the period. As the demand increases, the input from source PS1 increases in comparison to the other two sources. This happens despite it neither having the lowest tariff rate nor the best efficiency.

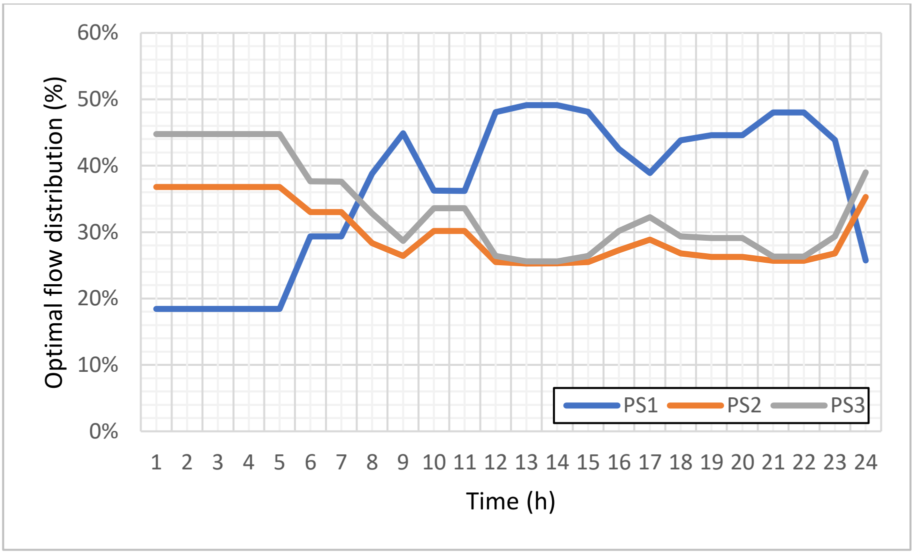

Figure 91. Optimal flow distribution in %

Recalling the objective function, the costs are given by the sum of the pumping costs and the water treatment costs. If only treatment fares were considered, the order of preference of pumping stations will be from PS3 to PS1 (Table 28). This is, PS3 will be the main source of supply and PS1 will be the last in importance. However, Figure 90 and Figure 91 show a different flow distribution. In that sense, the pumping preference is given to the pumping station PS1, then pumping station PS3 and finally pumping station PS2. Thus, the difference likely is due to the pumping costs. The pumping costs are computed in function of the energy tariffs, the expected pumping efficiency, the pumping time and finally the flow discharge and pressure head provided by the pumping station. At a first sight, energy tariffs are very similar, hence, attending more to the efficiency the flow distribution order should be PS2, PS3 and PS1. Besides, PS2 elevation is higher than the other pumping stations. But again, the assumptions do not fit with the results. At this point, it could be thought that the outcomes are suboptimal or that there was a sort of calculation mistake. However, up to now, both the flow and the pressure head to be supplied by the pumping stations have been left aside. To determine their influence, the Figure 92 has been plotted. The figure reflects the total operating cost of each pumping station separately regarding the percentage of flow supplied to satisfy a specific demand of the network. As a demonstration, it has been taken the demand multiplier of 2 which is reached between the 12:00 and 14:00 hours. Thus, a representation of the costs associated with each pumping station for the different flow distributions has been made. In that context, the figure shows that PS2 and PS3 are cheaper when the percentage of flow supplied by them is lower than 40% approximately. Following on from there, PS1 is more profitable. In fact, good solutions are reached when PS3 assumes percentages around 40% of the flow distribution. However, the cheapest solution belongs to a flow distribution of PS1 = 49%, PS2 = 25% and PS3 = 26%. Therefore, it can be noted that pumping station PS1 is not always the more expensive source of supply. From all that has been mentioned above, it can be said that cheap pumping conditions and even high values of efficiency, not always means that a pumping station must be preferred over all the other ones. Aspects such as the interaction with the network, the interaction with other pumping stations, and the placement of the pumping stations, are also important. This because of their direct influence over the variables of pressure head and flow to be supplied by the pumping stations. In that context, it can be thought that the optimisation methodology applied in this study leads to significant savings which can be difficult to infer at first sight.

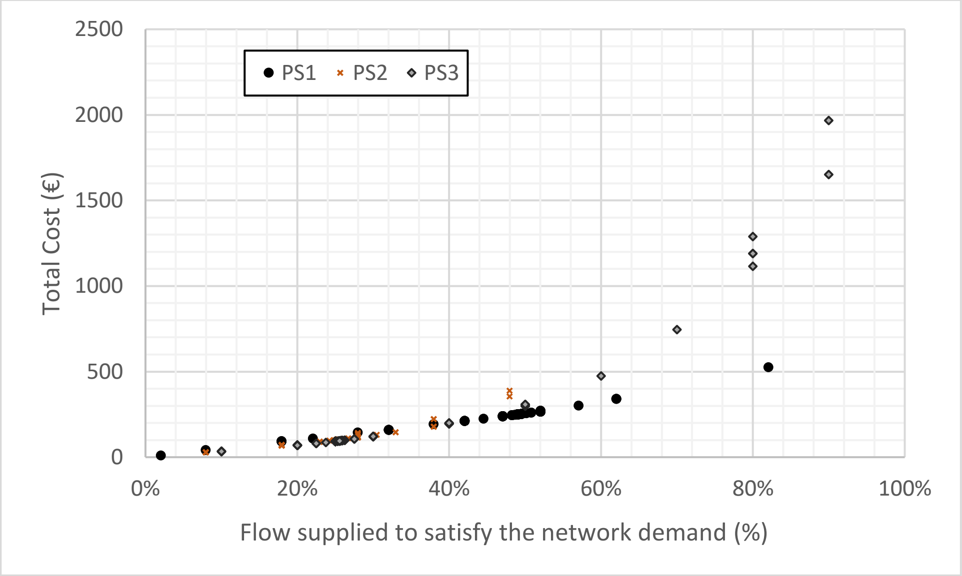

Figure 92. Total costs of pumping stations regarding the percentage of flow supplied when the demand multiplier is 2

The optimal SCs obtained for the three sources are shown in Figure 93. These curves can be very useful for the pumping system selection. In that sense, although pumps selection study is beyond of the scope of this work, it may result convenient to demonstrate the feasible of finding a pumping system that fit with the SCs obtained. Thus, next an example will be presented.

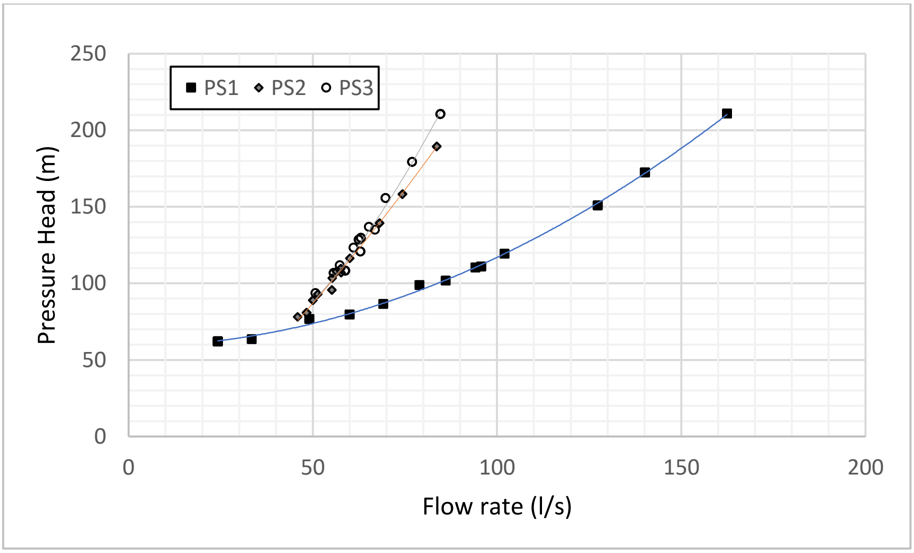

Figure 93. Optimal setpoint curves of TF network since cost optimisation approach

For the pumping selection a catalogue of 56 pumps will be used. Table 29 shows the information of the nominal flow rates, and pump heads. Besides, the manufacturer assumes a maximum efficiency $\eta_{max} = 80\%$ for each of its pumps.

All pump performance curves accomplish with the following expressions depending on whether they are FSPs (Equation 72 and 73) or VSPs (Equation 74 and 75):

$$H = a - c \cdot Q^2 \quad \textbf{(72)}$$

$$\eta = e \cdot Q - f \cdot Q^2 \quad \textbf{(73)}$$

$$H = a \cdot {\alpha_s}^2 - c \cdot Q^2 \quad \textbf{(74)}$$

$$\eta = \frac{e}{\alpha_s} Q - \frac{f}{{\alpha^2}_s} Q^2 \quad \textbf{(75)}$$

To calculate the pump performance curves in accordance with the optimum pump operation point (i.e. design flow rates and design pump heads, maximum efficiency), the following assumptions have been made:

$$Q_{max} = 2 \cdot Q_d \quad \textbf{(76)}$$

$$Q_{max} \to H = 0, \eta_{max} = 0 \quad \textbf{(77)}$$

$$\eta_{max} \to H = H_d, Q = Q_d \quad \textbf{(78)}$$

Table 29. Pumps catalogue. Nominal service point 2900 rpm

Model	Q_d (l/s)	H_d (m)	Model	Q_d (l/s)	H_d (m)	Model	Q_d (l/s)	H_d (m)
1	3.5	20	20	27.8	50	39	111.1	128
2	6.9	20	21	44.4	50	40	175	128
3	13.9	20	22	69.4	50	41	3.5	200
4	27.8	20	23	111.1	50	42	6.9	200
5	44.4	20	24	175	50	43	13.9	200
6	69.4	20	25	3.5	80	44	27.8	200
7	111.1	20	26	6.9	80	45	44.4	200
8	175	20	27	13.9	80	46	69.4	200
9	3.5	32	28	27.8	80	47	111.1	200
10	6.9	32	29	44.4	80	48	175	200
11	13.9	32	30	69.4	80	49	3.5	260
12	27.8	32	31	111.1	80	50	6.9	260
13	44.4	32	32	175	80	51	13.9	260
14	69.4	32	33	3.5	128	52	27.8	260
15	111.1	32	34	6.9	128	53	44.4	260
16	175	32	35	13.9	128	54	69.4	260
17	3.5	50	36	27.8	128	55	111.1	260
18	6.9	50	37	44.4	128	56	175	260
19	13.9	50	38	69.4	128			

Thus, from equations 72,73, 76-78 the parameters to build the pump performance curve and efficiency curve are obtained.

$$a = \frac{4 \cdot H_d}{3} \tag{79}$$

$$c = \frac{H_d}{(3 \cdot Q_d)} \tag{80}$$

$$e = 2 \cdot f \cdot Q_d \tag{81}$$

$$f = \frac{\eta_{max}}{{Q_d}^2} \tag{82}$$

The information of the pumps selection is presented in Table 30. The process has been made taking into account the critical points (i.e. maximum flow and pressure head) of the optimum SCs. On the other side, the number of pumps can improve the efficiency. Thus, the total of pumps has been chosen aiming to obtain higher efficiencies than the minimum expected. Regarding the models selected, PS1 and PS3 have the same model, though they require a different number of pumps. In the case of PS2, a different model is needed.

Table 30. Pumps selection for TF network

	PS1	PS2	PS3
Q (l/s) (SC)	162.40	83.62	84.64
H (m) (SC)	210.95	189.49	210.60
Nº pumps	3	2	2
Model selected	53	45	53
Q_d (l/s)	44.4	44.4	44.4
H_d (m)	260.0	200.0	260.0
a	346.666667	266.666667	346.666667
c	0.04396288	0.03381760	0.04396288
e	0.03603604	0.03603604	0.03603604
f	0.00040581	0.00040581	0.00040581

For PS1 a system of 3 pumps has been selected. Two of them are VSPs and the last one is a FSP. All the pumps have the same size. However, the number of pumps in operation will depend on the demand variation. Besides, when two VSPs are needed, their operating speed will be the same. The operating conditions of the pumping system are shown in Table 31. In Table 28 can be seen that the minimum efficiency expected at PS1 is 60%. In that sense, all pumps are working over the minimum value of the efficiency. To know the efficiency of the system, the global efficiency must be calculated. For that purpose, the next expression is used:

$$\eta_{global}(\%) = \frac{\sum_{i=1}^{24} Ph_i}{\sum_{i=1}^{24} P_i} \quad \textbf{(83)}$$

Where,

η_{global} is the global efficiency of the pumping system,
Ph_i is the hydraulic power, and
P_i is the power of the pumps.

As Table 31 shows, the global efficiency of PS1 is 71.35%. Therefore, this result points out that by using the selected pumps, efficiencies over the minimum expected can be obtained.

Table 31. PS1 pumping system

t (h)	Nº VSP	Nº FSP	VSP				FSP		Total	Total
			Q (l/s)	H (m)	α_s (%)	η (%)	Q (l/s)	η (%)	P (kW)	Ph (kW)
1	1	-	24.20	62.04	50.32	79.45	-	-	18.54	14.73
2	1	-	24.20	62.04	50.32	79.45	-	-	18.54	14.73
3	1	-	24.20	62.04	50.32	79.45	-	-	18.54	14.73
4	1	-	24.20	62.04	50.32	79.45	-	-	18.54	14.73
5	1	-	24.20	62.04	50.32	79.45	-	-	18.54	14.73
6	2	-	49.03	76.65	54.53	79.99	-	-	46.09	36.87
7	2	-	49.09	76.67	54.55	79.99	-	-	46.16	36.92
8	2	-	78.91	98.84	69.46	73.76	-	-	103.73	76.51
9	2	-	102.08	119.24	82.12	67.21	-	-	177.67	119.41
10	2	-	60.03	79.70	58.66	78.14	-	-	60.06	46.93
11	2	-	59.97	79.68	58.64	78.16	-	-	59.97	46.88
12	2	1	77.15	172.38	82.82	79.81	62.96	66.02	324.76	236.94
13	2	1	106.84	210.94	98.51	76.08	55.56	74.94	444.01	336.06
14	2	1	106.84	210.95	98.51	76.08	55.56	74.94	444.02	336.07
15	2	1	77.23	172.43	82.86	79.80	62.95	66.03	324.99	237.13
16	2	-	86.08	101.80	72.70	71.11	-	-	120.89	85.97
17	2	-	69.16	86.54	63.35	75.79	-	-	77.47	58.71
18	2	-	94.15	110.25	77.40	69.06	-	-	147.45	101.83
19	2	-	95.70	110.94	78.13	68.48	-	-	152.08	104.15
20	2	-	95.69	110.94	78.12	68.49	-	-	152.07	104.15
21	2	-	127.34	150.91	97.44	62.20	-	-	303.11	188.53
22	2	-	127.35	150.91	97.44	62.20	-	-	303.12	188.53
23	2	-	94.18	110.26	77.41	69.04	-	-	147.55	101.87
24	1	-	33.43	63.54	57.01	71.78	-	-	29.03	20.83
							Total (kW)		3556.89	2537.92
							η_{global} (%)		71.35	

In the case of PS2, the pumping system is formed by two VSPs of the same size (30). Both pumps work at the same speed over the whole simulation period (Table 32). The minimum efficiency expected is 75% (Table 28). In that sense, all the efficiency values are over the minimum expected. In fact, the global efficiency is 79.43% which is very close of the maximum efficiency of the pumps.

Table 32. PS2 pumping system

t (h)	Nº VSP	Q (l/s)	H (m)	α_s (%)	η (%)	P (kW)	Ph (kW)
1	2	48.29	80.89	61.42	78.95	48.54	38.32
2	2	48.29	80.89	61.42	78.95	48.54	38.32
3	2	48.29	80.89	61.42	78.95	48.54	38.32
4	2	48.29	80.89	61.42	78.95	48.54	38.32
5	2	48.29	80.89	61.42	78.95	48.54	38.32
6	2	55.17	95.79	67.51	79.49	65.22	51.85
7	2	55.17	95.79	67.51	79.49	65.22	51.84
8	2	57.69	107.01	71.19	79.39	76.29	60.57
9	2	60.03	116.33	74.20	79.37	86.32	68.51
10	2	49.97	88.99	64.26	78.77	55.38	43.62
11	2	50.04	89.08	64.30	78.78	55.50	43.72
12	2	74.34	158.32	87.69	79.84	144.62	115.45
13	2	83.56	189.36	96.51	79.95	194.14	155.22
14	2	83.62	189.49	96.56	79.95	194.43	155.45
15	2	74.28	158.21	87.65	79.83	144.40	115.28
16	2	55.20	103.41	69.60	79.09	70.80	55.99
17	2	51.29	93.00	65.74	78.82	59.37	46.79
18	2	57.59	109.67	71.86	79.24	78.19	61.95
19	2	56.35	107.85	71.07	79.08	75.40	59.62
20	2	56.40	107.91	71.10	79.09	75.49	59.70
21	2	68.10	139.49	81.86	79.68	116.95	93.19
22	2	68.10	139.49	81.86	79.68	116.95	93.19
23	2	57.50	109.54	71.81	79.23	77.99	61.79
24	2	45.88	78.07	59.96	78.47	44.78	35.14
				Total (kW)		2040.15	1620.51
				η_{global} (%)		79.43	

Pumping station PS3 has the same number and type of pumps than PS2. Though, the size of the pumps is different (30). The VSPs work at the same speed over the simulation period. Table 33 shows that for the supplied flow the pumps work over the minimum efficiency expected (65%, Table 28) and with a global efficiency of 79.95%.

Table 33. PS3 pumping system

t (h)	Nº VSP	Q (l/s)	H (m)	α_s (%)	η (%)	P (kW)	Ph (kW)
1	2	58.75	108.18	64.92	79.97	77.96	62.35
2	2	58.75	108.18	64.92	79.97	77.96	62.35
3	2	58.75	108.18	64.92	79.97	77.96	62.35
4	2	58.75	108.18	64.92	79.97	77.96	62.35
5	2	58.75	108.18	64.92	79.97	77.96	62.35
6	2	62.94	120.83	68.86	79.93	93.34	74.60
7	2	62.87	120.69	68.81	79.93	93.12	74.44
8	2	66.83	135.04	72.88	79.91	110.78	88.53
9	2	65.23	136.79	72.77	79.99	109.42	87.53
10	2	55.70	106.85	63.76	79.98	72.99	58.38
11	2	55.70	106.85	63.76	79.98	73.00	58.39
12	2	76.97	179.32	83.97	79.92	169.42	135.39
13	2	84.64	210.60	91.36	79.85	218.98	174.86
14	2	84.57	210.44	91.31	79.85	218.64	174.59
15	2	76.94	179.25	83.95	79.92	169.29	135.29
16	2	61.04	123.20	68.81	80.00	92.22	73.78
17	2	57.31	111.71	65.30	79.99	78.52	62.81
18	2	63.04	129.65	70.71	80.00	100.23	80.18
19	2	62.50	128.53	70.33	80.00	98.51	78.81
20	2	62.46	128.46	70.30	80.00	98.40	78.72
21	2	69.74	155.73	77.68	79.99	133.19	106.54
22	2	69.74	155.72	77.68	79.99	133.18	106.53
23	2	63.09	129.74	70.74	80.00	100.37	80.29
24	2	50.69	93.53	59.27	79.89	58.21	46.51
				Total (kW)		2611.63	2087.90
				η_{global} (%)		79.95	

From the pumps selection, it can be noted that a pumping system that fit the optimal SCs has been proposed. Besides, the pumping system works with higher efficiencies than the minimum values expected. However, it must be highlighted that the pumping system selected is not optimal since better solutions can be found. In that sense, the number of pumps, the size, the operation, could be changed. Moreover, other alternatives such as the use of hydropneumatic drums, valves, by-pass lines, pumps with different sizes, etc., could be studied. All these options may lead to different costs both investment and operational. In that sense, it may result interesting to make a life cycle cost analysis of different solutions to define the optimum one. However, all aforementioned requires a more comprehensive research work which is out of the limits of this document.

It should also be mentioned that the minimum pressure in the network over the whole simulation period is implicitly satisfied and guaranteed as part of the SC calculation process. Hence, add another search objective or a constraint to meet this goal is not necessary. As PDDs are considered, it is essential to keep the minimum pressure on the network, so use the SC can mean significant savings regarding the water demand.

With the information collected (i.e. optimal flow distribution and least cost SCs) not only an operating cost optimisation of pumping can be done but also an analysis of the energy cost influence over the flow distribution. This in order to simulate the impact of the network management when different flows distributions among pumping stations are tested.

5.3.2. COPLACA network

This network has been already introduced in section 4.5.3. (Figure 84). Thus, the network has been already optimised from the energy approach where results show that only five water sources are required to satisfy the demand of the system (Figure 86). However, starting from the cost optimisation approach an additional assessment will be developed.

The efficiency and the cost of water treatment of each source are presented in Table 34. They are assumed to be constant over time, as it was done in the previous case.

Table 34. Q_{max}, Q_{min}, performance and water treatment cost of the water sources of the network (COPLACA network).

Sources	P05	P06	P07	P11	P12	P13	P10
Efficiency (%)	60	75	65	70	80	60	70
Water treatment cost [€/m³]	0.34	0.38	0.36	0.27	0.30	0.30	0.25
Q_{max} [l/s]	9.0	3.0	7.0	17.0	15.0	15.0	80.0
Q_{min} [l/s]	0.5	0.5	0.5	0.5	0.5	0.5	0.0

The demand curve was obtained for 24 hours (Figure 94). As in the previous case, the energy tariffs have an hourly discretisation divided into four periods (Table 35).

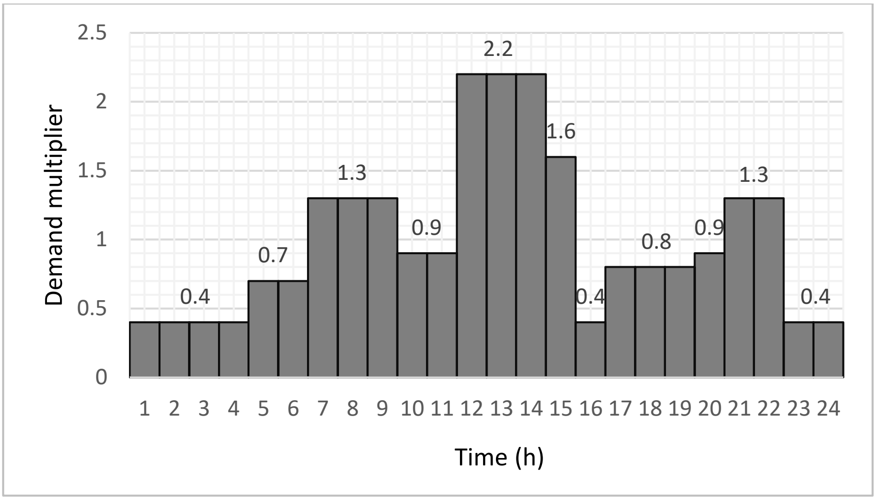

Figure 94. Demand variations of COPLACA network

Table 35. Energy tariffs regarding the time and water source (COPLACA network)

Time (h)	P05, P07, P13 (€/kWh)	P06, P11 (€/kWh)	P12, P10 (€/kWh)
1-8	0.09	0.092	0.094
9-18	0.129	0.131	0.132
19-22	0.162	0.164	0.165
23-24	0.129	0.131	0.132

In this case, it can be seen (Figure 95) that for most source the SCs are flat and some of those collapses into a single point. The SC for P10 is the only one spread over the range of flows. This information can be used to support decisions on how to regulate the pumping systems for each water source, i.e., whether it needs variable speed regulators (e.g., P10) or can be kept as fixed-speed pumps. On the other hand, optimisation (Figure 96) shows that all sources are required to work together only during the peak demand periods. Therefore, the results can lead to a better water management plan, including maintenance and operation plans, because just some of the water sources are required to meet demand at certain times of the day. As minimum pressures are also maintained, the consumption is kept low as it is pressure dependent. The river source (P10) has not reached its maximum capacity (Figure 95), thus allowing to eliminate the need for additional sources. Hence, the existing network has enough capacity to meet the daily demand.

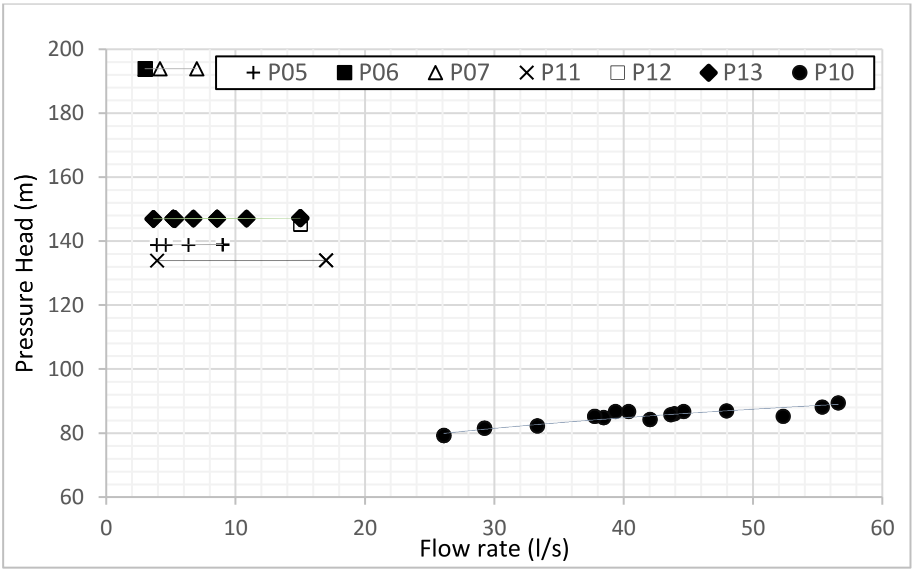

Figure 95. Optimal setpoint curves of COPLACA network

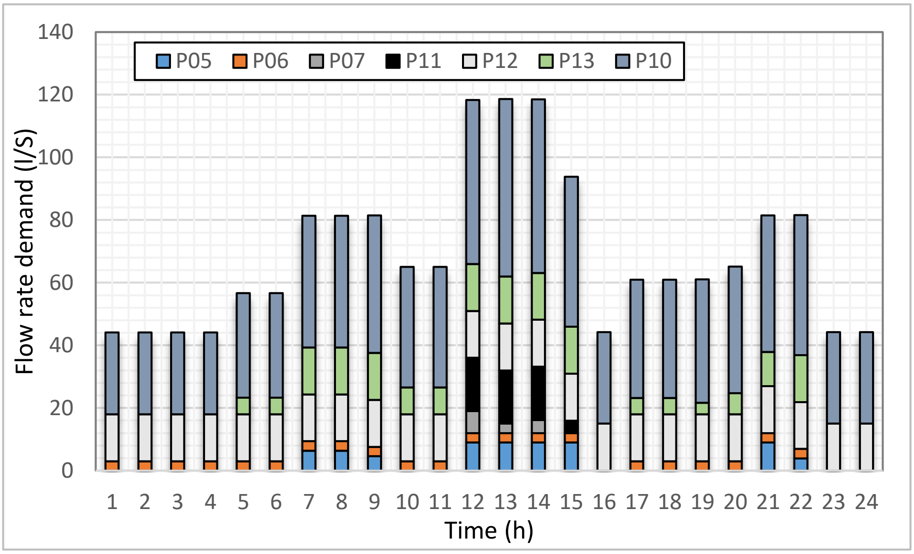

Figure 96. Optimal flow distribution of COPLACA network over the time

It is worth noting that if the source production costs are considerably higher than the energy costs and the differences among them are significant, the flows from different sources will be distributed mainly according to those rates, i.e. from the lowest to the highest cost. In that case, the energy cost may be less important. Therefore, optimisation may arrive at an obvious answer. However, through the methodology presented it is still possible to find the least-cost flow distribution to be supplied by the different water sources.

When results that were obtained from the energy optimisation approach (see, section 4.5.3.) are compared with results obtained from the cost optimisation approach then, it can be noted that costs influence both the range of flow rates to be supplied by each pumping station as the number of pumping stations that participate in the optimal distribution. This without even propose the sizing or selecting any pump but considering the optimal work conditions of the network. In that sense, some pumping stations that before were thought without value from the energy saving point of view, now are essential from the operating costs point of view.

Ending the cost optimisation, it can be observed that the formulation of the problem starts from the C-M of the previous chapter. This means the optimal flow distribution is done by assuming the flows as continuous variables. However, this time energy fares and other costs are also included in the objective function. Another aspect taken into account is the expected efficiency of the pumping stations which has an important role in the optimisation process. This way, optimal pumping costs for the different flow distributions are computed. Regarding the cases study, it can be noted the relative importance of pumping stations. Though its operation is linked to the energy consumption, facilities, network demand, efficiency, among other important features, it can be noted the special relevance of the energy costs. In that sense, for the same network demand in different periods of the day, the flow distribution among pumping stations can be totally different. Thus, it is evident that cost optimisation can affect substantially not only the pumping operation but also the perception of the water supply sources relevance.

Chapter 6
Energy and cost optimisation with storage capacity

So far, it has been addressed the pumping optimisation through getting the optimal setpoint curves (SCs) of pumping stations from two approaches:

a) first starting from the least-energy demanding SCs (i.e. pumping energy optimisation), and then
b) by calculating the least-cost SCs through including both energy fares and treatment costs within the analysis (i.e. cost optimisation).

In this context, it is desirable to remember some of the assumptions previously established or intrinsic to the optimisation process studied up to now (i.e. Chapter 4 and 5):

a) The values of pressure head and flow necessaries to draw the optimal SC are found by using a static hydraulic model. The aim is to determine the optimal flow distribution that meets the demand while the minimum pressure at the critical node is kept. The optimal flow distribution is obtained for each time step. Thus, the process is repeated when the network demand changes to find more points of the SCs.
b) The pressure head at the critical node must be equal to the minimum pressure required over the whole period of simulation. This condition has to be accomplished independently of the network demand changes as well as the variations of flow distributions among the pumping stations.
c) A dummy reservoir is needed to adjust and keep constant the pressure at the critical node.

d) The summation of the inflow optimal flow distribution in the period *i* is equal to the network demand at the same time *i*.
e) Both constraints, the quantity of flow that a pumping station can supply as well as the condition of minimum pressure that must be kept at the critical node, are indirectly considered within the setpoint calculation process and do not need to be added to the OF.
f) Only it is needed a direct search algorithm to carry out the optimisation.

In the last part of the research, the storage capacity of the network (i.e. tanks) is included. Therefore, all the previously enumerated assumptions cease to be valid either partially or totally. In that sense, a new approach to considered least-cost SCs in networks with storage capacity is formulated.

When there is storage capacity, the analysis of the hydraulic model will be quasi-static and will be developed in an extended period. Besides, the dummy reservoir used to adjust the pressure of the critical node, cannot be longer applied. The reason is that the pressure head at critical node will be inevitably above the value of the minimum pressure required and will be unknown because of two possible scenarios:

a) whenever it is required to fill the tanks since they are in higher points than critical nodes and must operate within pre-specified storage levels, and
b) when the pressure head of the network is governed exclusively by the tanks, i.e. when pumps are not working.

Thus, all pumping stations are represented as inflow nodes with negative demand and the dummy reservoir is not considered in this approach.

The formulation of the SC concept exposes that the curve is given by the addition of three terms, the static lift, the head loss in the network elements, and the minimum pressure required (Equation 58). In the explained methodologies for both energy and cost optimisation of networks without tanks, optimum SC is formed by isolated points (i.e. the most economic values of pressure and flow) got from different SCs, as many as tested flow distributions. Thus, when there is an irregularity in the shape of the SC, this is due to the variation introduced in head loss term as a consequence of the different flow distributions among pumping stations. Another cause is the variation of the static lift when the location of the critical node changes, though the minimum pressure is kept constant. In any case, most of the time the optimal SC has a shape of a parabola. However, when tanks are considered, new variations on the three terms that constitute the SC formulation are introduced. In that sense, tanks levels trajectory affects the static lift of pumping stations, the head loss due to the flow changes and the minimum pressure as it was already exposed. Therefore, there will be as many SCs as changes static lifts, flow distributions, and minimum pressures variations. In that context, the optimal SC will be formed by isolated points got from those curves. However, as there are a major number of SCs to consider, the variation between the obtained points will be bigger and also the irregularities in the final shape of the optimal SC.

On the other hand, the pumping stations flow rate has to be enough to meet not only the consumption of the network but also to fill the reservoirs over the whole period of simulation. The quantity of water to be supplied by each pumping station can be stated as follows:

$$Q_{ij} = -X_{ij} \cdot W \cdot QMD \quad \textbf{(84)}$$

Where,

Q_{ij} is the flow rate of the pumping station *j* at simulation period *i*,
X_{ij} is the variable of decision that defines the quantity of flow rate to be supplied by the pumping station *j* at period *i*, (any value between 0 and 1),
W is a constant value that points out the peak flow over the whole simulation period,
QMD is the average flow demand of the network.

It could be thought that W corresponds to the peak factor of the average daily flow demand. However, it may happen that in a critical situation the pumping stations besides satisfying the peak demand of the network must supply the storage tanks. Thus, the value of *W* can be estimated as two or three times the average daily flow.

If there is any flow restriction either concerning the maximum or minimum flow rate that the pumping station is capable of supplying, the flow allocation shall be expressed by Equation (85).

$$Q_{ij} = -\left[Qmin_{ij} + X_{ij} \cdot \left(Qmax_{ij} - Qmin_{ij}\right)\right] \quad \textbf{(85)}$$

Where,

$Qmin_{ij}$ is the minimum flow rate of the pumping station *j* at period *i*, and
$Qmax_{ij}$ is the maximum flow rate of pumping station *j* at period *i*.

The allocation of flow distributions among pumping stations is done for the entire horizon of simulation since the beginning. Therefore, as many flow distributions as time steps of the extended period have to be generated. This means that before solving the network model, the supply flows must be assigned to each pumping station *j* for the total number of scenarios. In that sense, there is a significant difference regarding the other methodologies presented in the research where the optimisation is performed for a permanent regime (i.e. the optimisation is carried out for each time step separately). Moreover, the elevation of the inflow nodes will be given by the piezometric head available in the suction of the pumping stations, so when the hydraulic model is solved, the pumping heads corresponding to each pumping station will be obtained directly.

Also, as tanks must be operated within a prespecified range of levels, i.e. minimum storage level ($Lmin_{ta}$) and maximum storage level ($Lmax_{ta}$), the optimisation of the initial levels may be needed. For that, the same variable of decision is applied, $X_{ij} \in (0,1)$. In this context, the variable of decision has two meanings: the flow supplied by pumping stations and the initial levels of the tanks. The initial level of each one of the ta tanks will be formulated as follows:

$$L_{ta} = Lmin_{ta} + X_{ij} \cdot (Lmax_{ta} - Lmin_{ta}) \qquad ta = 1, \dots \dots, TT \tag{86}$$

Where,

TT is the number of tanks.

Thus, the number of decision variables X_{ij} will not only given by the number of pumping stations (*Nps*) and the number of periods of simulation (*Nst*), but also by the number of tanks (*TT*). This is, the number of dimensions (*ND*) of the problem will be given by the next expression:

$$ND = Nps \cdot Nst + TT \tag{87}$$

Once the flow rates have been assigned, the hydraulic model is solved, and the pressure head at the critical node of the network is determined at each time *i*. Subsequently, it must be verified that the obtained pressure is equal to or higher than the minimum pressure required. Also, it will be checked that the final levels in the tanks are similar to or higher than the initial levels. These two constraints will be assessed directly as a part of the OF as will be shown in the next section. Thus, the flow distribution and tank levels (Equations 84 and 85) shall be adjusted to their optimum values avoiding breaching pressure restrictions and storage levels recurrence.

Considering the aforementioned and before the application of the formulated approach the following premises must be fulfilled:

a) The minimum pressure required is achieved at all nodes including of course the critical node.
b) Pumping stations are not defined (i.e. there is no need for specifying the number of pumps or the pumps performance curves).
c) All pumping stations are represented as inflow nodes.
d) Tanks are located at a point high enough to guarantee the needs of pressure head at the network.
e) There are at least one tank and one pumping station in the system.

In the case of a network with booster pumps, the system is split in each point where the booster pumps are located. This way two nodes instead of one will be used to represent the booster stations. One will be the suction node, and the other will be the discharge node. Hence, to obtain the required pressure head at the pumping station, the difference of piezometric heads between the two nodes must be determined. Despite there are two nodes, the value of the decision variable X_{ij} is only one, since the demand of the node A will be the inflow of the node B (Figure 97). In the case that the HGL elevation available be major than the HGL elevation required, the booster pumping station will not work. Therefore, optimal solutions should tend to find equal values for the HGL elevations at suction and discharge. This way, lower energy will be used by the pumping system.

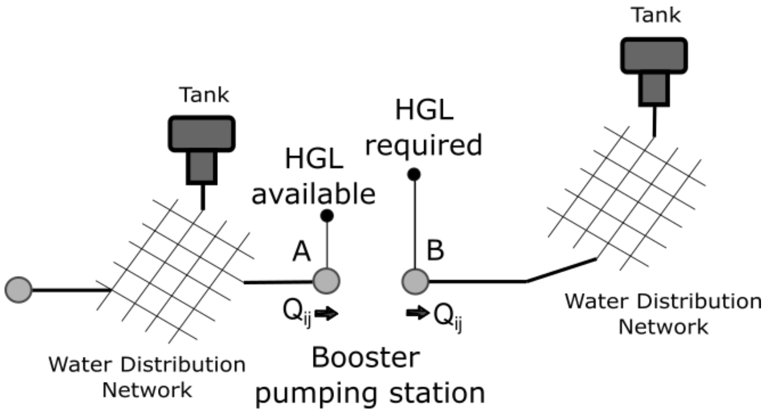

Figure 97. Booster pumping station in networks with tanks

It has to be highlighted that when the network has no tanks, the number of dimensions of the problem is given by the number of pumping stations, but when tanks are considered, the number of dimensions increases at least *Nst* times (e.g. 24 times when the optimisation is developed for a complete day). The number of dimensions also increases when the initial tank levels are optimised, see Equation 87. Thus, the search space is much bigger. In that context, direct search algorithms applied up to now (H-J and N-M) have a high risk of being trapped in local optimum values. This because they tend to reduce the search space when a good solution is obtained. To avoid problems with local optimum an evolutive algorithm will be used. In this case either, Differential Evolution (DE) [12] algorithm which has been already introduced, or a new hybrid algorithm developed as a part of the present research which will be explained later. Since the objective of the study is to obtain the lowest energy cost, the evaluation of the discharge flows will be carried out by minimising the cost of the OF, as indicated in the following section.

6.1. Objective Function

The OF has been formulated taking into account the sum of the minimum cost of its four terms. The first one corresponds to the *pumping energy cost (PEC)*, the second one to the *treatment water cost (TWC)* and the last two are penalty costs related with the pressure *(PPC)* and with the volume of the reservoir tanks *(VPC)*. It has to be kept in mind that capital costs are no part of this study as was explained in the previous chapter.

$$Min\, f(X) = PEC + TWC + PPC + VPC \quad \textbf{(88)}$$

The *pumping energy cost* can be calculated by the sum of each one of the energy costs related to the different pumping systems. Each pumping systems corresponds either to a water source or booster pumps. As an extended period of analysis is carried out, the final value will depend on the variations of the flow rates and pressure heads of SCs of each pumping station as well as the energy tariffs and pumping time over the simulating period. The simulation periods will be given by the network demand changes and the state change at tanks.

$$PEC = \sum_{i=1}^{Nst}\sum_{j=1}^{Nps}\frac{\gamma \cdot Q_{ij} \cdot PH_{ij}}{\eta_{ij}} \cdot ET_{ij} \cdot t_i \quad \textbf{(89)}$$

Where,

PEC is the sum of pumping energy cost of each pumping system *j* at the end of the simulation period,
Nps is the number of water supply sources (pumping stations),
γ is the specific weight of the water,
Q_{ij} is the flow rate obtained from the SC (Equation 84, 85) of pumping station *j* in the simulation period *i*,
PH_{ij} is the pressure head from the SC needed to deliver a specific flow rate by the pumping station *j* over the simulation period *i*,
η_{ij} is the expected efficiency of the pumping station *j* at the simulation period *i*,
ET_{ij} is the electric tariff corresponding to the pumping station *j* over the simulation period *i*,
t_i is the pumping time at the simulation period *i*.

It should be taken into consideration that no pumps have been selected yet. Thus the pumps system is not defined. For that reason, the efficiency of each pumping system is assumed as a fixed value. Although the efficiency topic was explained before, it is worth to address it again. The efficiency value considered will be an estimation of the expected minimum efficiency of the pumping station once all the pumps have been selected. Thus, after optimal SCs are got, the next step will be select a pumping system that is capable of satisfying at least the minimum efficiency assumed. This stage is out of the scope of the research. On the other hand, when the efficiency value is taken as the maximum expected efficiency, sub-optimal solutions are got because pumps cannot work at maximum efficiency all the time. However, it may be interesting suppose the maximum efficiency as a way to get an idea of the maximum possible savings in ideal work conditions. In this way, it is possible to know how far it is possible to get in terms of optimising operating costs.

The *treatment water cost* is referred to the energy operation costs to treat the water, chemicals, additional pumping costs, and others which are needed to provide the water until the point where it will be pumping to the network. It is important to consider that these costs can also influence the use of one pumping system or other. All those costs are concentrated in a unique value defined as unit treatment costs (TC_{ij}). The final cost will be given by the sum of the product between the pumping water and its TC_{ij}.

$$TWC = \sum_{i=1}^{Nst}\sum_{j=1}^{Nps}\left(Q_{ij} \cdot TC_{ij} \cdot t_i\right) \tag{90}$$

Where,

TWC is the sum of the water produced cost of each pumping system *j* at the end of the simulation period;

TC_{ij} is the water production tariff of pumping station *j* at the simulation period *i*.

The next term corresponds to the cost due to non-compliance of the pressure constraints at consumptions nodes of the network. For that, the difference between the pressure in each node of the network and the minimum pressure required are compared.

$$PPC = \sum_{i=1}^{Nst}\sum_{n=1}^{TN}\left|PH_{i,n} - PH_{min}\right| \cdot K_{1,i,n} \cdot \lambda_{1,i} \tag{91}$$

Where,

PPC is the pressure penalty cost calculated as the sum of the penalty cost for each node *n* at the end of the simulation period,

TN is the number of total consumption nodes of the network,

$PH_{i,n}$ is the pressure of the node *n* at simulation period *i*,

PH_{min} is the minimum pressure head required on the network over the simulation period *i*,

$K_{1,i,n}$ is a temporal factor which appears as long as the pressure head at node *n* and simulation period *i* is lower than the minimum pressure head required.

As long as the pressure is less than the minimum pressure needed for the network an additional cost is added to the OF. Only those nodes that not accomplish with the restriction are considered. For that reason, the difference in pressure is multiplied by a factor K_1 which can take either a value of zero, if the pressure on the node is equal or superior to the minimal pressure required, or a value of one when the pressure restriction is not met. It can be described as follows:

$$\begin{aligned} &If\ PH_{i,n} - PH_{min} < 0\ then\ K_{1,i,n} = 1 \\ &If\ PH_{i,n} - PH_{min} \geq 0\ then\ K_{1,i,n} = 0 \end{aligned} \quad \textbf{(92)}$$

In order to compute the *PPC*, it will be enough to identify only the critical node. However, in complex networks there could be more than one critical node. Thus, the search algorithm improves its efficiency when all nodes are considered. To convert the pressure difference into a cost it is also used a factor $\lambda_{1,i}$. The factor $\lambda_{1,i}$ could be assumed as a constant value (e.g. $\lambda_{1,i} = 1 \cdot 10^6$). However, this can be too much restrictive for the OF and good approximations to the final solution could be discarded. These solutions can later become into good solutions while the OF is optimised.

$$\lambda_{1,i} = \frac{\gamma \cdot Q_{i,n} \cdot Max(ET_{ij})}{min(\eta_{ij})} \cdot t_i \quad \textbf{(93)}$$

Where,

$Q_{i,n}$	is the demand of node *n* at simulation period *i*,
$Max(ET_{ij})$	is the maximum energy tariff over the whole simulation period, and
$min(\eta_{ij})$	is minimum value of the efficiency among all the pumping stations.

The last term in Equation (88) is the cost due to non-compliance of the volume constraints in all the reservoir tanks available in the network. Further, it has been considered that when more water is required to meet with the optimum operating levels at the tanks, it is also necessary to produce more water. Therefore, the cost of water production has been added to the volume penalty cost as a second term.

$$VPC = \sum_{ta=1}^{TT} \left(\lambda_2 \cdot K_{2,ta} \cdot \sum_{i=1}^{Nst} \frac{V_{ta,i}}{t_b} \right) + \sum_{ta=1}^{TT} \left(K_{2,ta} \cdot \sum_{i=1}^{Nst} V_{ta,i} \right) \cdot Max(TC_{ij}) \quad \textbf{(94)}$$

Where,

VPC	is the total volume penalty cost that results in the sum of the penalty cost of each tank *ta* at the end of the analysis period,
$K_{2,\text{ta}}$	is the temporal coefficient that affects the costs depending on if particular conditions are accomplished,
$V_{ta,i}$	is the volume that goes in and goes out of the tank *ta* at the simulation period *i*,
t_b	is an assumed pumping time value required for eliminating the volume deficit,
$Max(TC_{ij})$	is the maximum water production tariff over the simulation period.

The penalty cost is considered as long as the volume at the end of the simulation of each tank does not meet specific requirements defined previously. In this case, the requirement obeys the fact that the sum of the volume from each tank *ta* must be equal or bigger than zero at the end of the analysis period. In this way, the tank level will be the same or bigger at the end of the simulation period. For that, a temporal factor $K_{2,ta}$ is needed. This factor only works when the sum of the volume in the tank *ta* is negative.

$$\begin{aligned} &If\ \sum_{i=1}^{Nst} V_{ta,i} < 0 \text{ then } K_{2,ta} = 1 \\ &If\ \sum_{i=1}^{Nst} V_{ta,i} \geq 0 \text{ then } K_{2,ta} = 0 \end{aligned} \quad \textbf{(95)}$$

The conversion of volume into cost is achieved through the factor λ_2.

$$\lambda_2 = \frac{\gamma \cdot Max(PH_{ij}) \cdot Max(ET_{ij})}{min(\eta_{ij})} \cdot t_b \quad \textbf{(96)}$$

Where,

$Max(PH_{ij})$ is the maximum pressure head among the pumping stations over the simulation period.

In a general way, the OF has been formulated taking as reference two aspects:

a) the hydraulics are solved in an extended period simulation (i.e. network demands, energy fares, tank levels, and others depend on the time), and
b) the SC calculation process is different to the methods where tanks are not considered.

Thus, the OF assess has to be done for the whole period simulation, and pressure and volume constraints have been added to the OF as a penalty cost terms. Following on from there, the next step consists into getting the values of flow and pressure head of pumping stations. For that, an optimisation method has to be applied. This is explained in the next section.

6.2. Optimisation Method

The assessment of the OF requires using a search algorithm. Since the number of variables of the problem is quite high (i.e. flow rates per pumping station and per period of simulation) it is recommendable the use of more powerful algorithms than the direct search methods applied before (i.e. H-J, and N-M). In that sense, both the minimum function cost as well as the optimal SCs will be got through the use of either DE algorithm or the Hybrid Algorithm. The DE algorithm has been already introduced in the section of optimisation algorithms. However, the Hybrid Algorithm is a contribution of this work as an attempt to reduce the time-consuming of the optimisation because of the high number of variables that the minimisation costs process involves. Both algorithms have been applied to the study of the cases that will be presented later. However, depending on the network size it can be more benefit (i.e. in terms of time) to applicate one or other. Before the optimisation process is addressed, the Hybrid Algorithm will be presented.

6.2.1. Hybrid algorithm

In the section corresponding to the revision of the DE algorithm, the advantages of the algorithm when it is required to deal with a large number of variables have been already mentioned. Other advantages are its programming simplicity and its ability to overcome problems with local optimum values which is its most important strength. However, as the algorithm has been applied to more complex distribution networks with a more significant number of dimensions (i.e. more pumping stations, more pipelines and nodes, the inclusion of the storage capacity of the system, extended period analysis) the calculation time has increased considerably. This has affected the efficiency of the algorithm negatively. In this way, when the efficiency of the DE algorithm is analysed, two stages can be distinguished. The high-efficiency stage where there is a great diversity of the population and the algorithm is relatively fast to discard non-feasible solutions and going to more convenient search areas. And, the low-efficiency stage, where the diversity of the population decreases, and the algorithm tends to stagnate. In that sense, in the second stage only is possible to reach a better solution after a big number of iterations is developed. Thus, there is the need to optimise the efficiency of the DE algorithm in order to obtain the global optimum and decrease the calculation time.

Based on the experience gained in the application of direct search algorithms (i.e., H-J and, N-M), it is known that these methods use the greedy criterion to accept or reject the solution vectors and its variations. Under this rule, the new vector is allowed considering if it reduces the value of the cost function. Thus, greedy criterion makes the convergence of the method occur faster. However, the huge drawback is that the algorithm is usually trapped in local minimums. In that sense, it is preferable the use of evolutive algorithms like DE as was previously introduced. Though, it has been observed that whenever a direct search method is applied to the best solution obtained by the DE algorithm, most of the time it is possible to improve that solution in somehow. Besides the local search operation is very fast. Therefore, taking into account that the DE algorithm aims to evolve the population until the optimum solution has been reached, the idea of finding better individuals starting from the population of the DE algorithm by applying a direct search method has been conceived. In this particular case, H-J algorithm will be used. At this point arise some problems that need to be solved. First of all, it is clear that it is impractical to use the H-J method to improve each element of the population of the DE algorithm because the computation time will increase rather than decrease. Neither it can be applied for each iteration of the algorithm since it would cause the same adverse effect in the calculation time. Therefore, criteria should be established about when the direct search algorithm will be activated and for what elements of population.

Of course, the idea of combine global search algorithms with local search algorithms is not new [60]–[62]. These kinds of algorithms are defined as Memetic Algorithms (MAs), where the local search is implicit within the global search aiming to get the advantage of both types of algorithms. However, variations between each method lie in the different strategies of application of the local search methodology. Usually, three aspects are considered, the exploitation area, application frequency and type of replacement [60]. The exploitation area refers to the kind of method used to carry out the mutation within the DE algorithm. The application frequency is about when is going to activate the local search. The usual criterion is to apply the local search operator to the best solution of the population when it does not improve after a certain number of generations, and it is overcome a specific value of probability. The value of the likelihood depends on there is a high or low diversity of population [63]. Finally, the last criterion is about what to do with the information obtained from the local search. Some methods replace a random vector, avoiding change the best solution which was used as starting point since the resulting vector could be worst. In that context, the methodologies developed are directed to give more fluency to the global search when it is not possible to find a better solution. And local search only is applied when global search slows.

Taking as a reference the three areas that difference one method of each other, the proposed algorithm presents some differences. For the exploitation area, it will be applied Rand-1 expression (Equation 16) since it has been observed experimentally through the simulations developed that is more effective for the type of problem studied. In the application of frequency appears the first difference. In this case, the local search will be applied to every better solution that improves the best current value of the function found by the DE algorithm. This as long as a predefined value, named as local search limit (LS), has been overcome. This value corresponds to the minimum number of iterations that the best value of the population remains invariable. However, local search only is applied if the global search finds a better solution. The type of replacement always will be given by the substitution of the prior best solution. Thus, the new better solution will have been optimised once time by the global search and a second time by the local search. In the case that better results are not found, the search process keeps the element used to perform the local search. In that sense, sometimes local search will not lead to better results, but not worse either. It must be considered that the aim of the local search is improved the individuals of the global search but not to find the global optimum. Therefore, the parameters of the local search method have to be adjusted to spend the minimum time possible before to improve the solution of the global search. A scheme of the Hybrid Algorithm is shown in Figure 98.

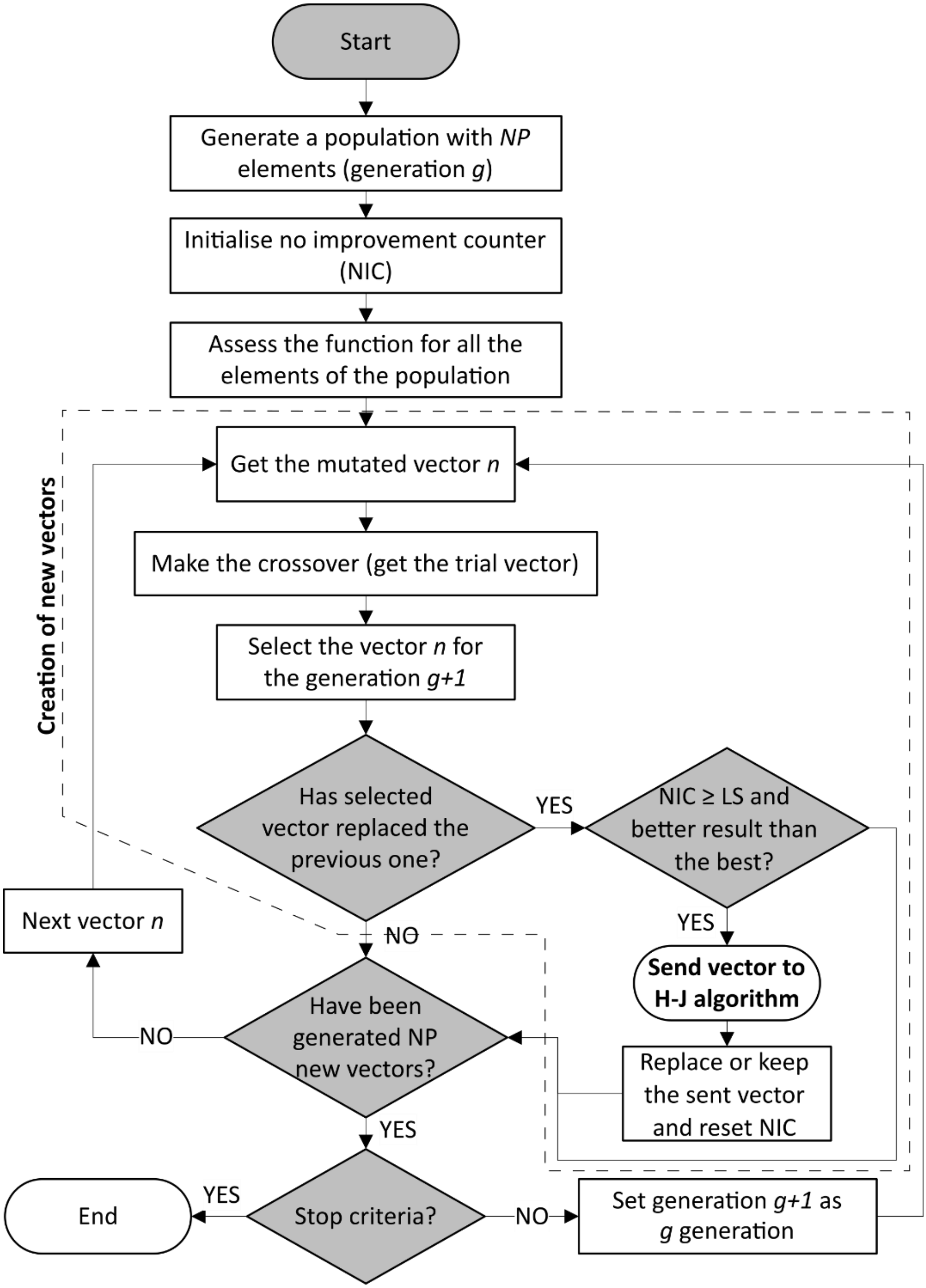

Figure 98. Scheme of the Hybrid Algorithm

Both algorithms, DE and H-J have been already presented (see, sections 2.3.1. and 2.3.3.). Hence, there is no need to explaining again the steps that each one of the algorithms follows. In that sense, the steps that define the Hybrid Algorithm pretend to show where and how the DE algorithm goes to the H-J algorithm and vice versa. Those steps are listed next:

1. Randomly generate the generation g of the population of vectors $(NP = \vec{X}_{n,g}, \dots, \vec{X}_{NP,g})$ and initialise the no improvement courter (NIC).
2. Calculate the fitness of each vector of the population
3. **While** stop criterion = false **Do** steps 4 **to** step 18
4. **For** *n = 1* **to** *NP* **do** step 5,6
5. **For** *j = n+1* **to** *NP* **do** step 6
6. Sort the values of the function and the vectors. **If** $F_j > F_n$ **then** swap $\vec{X}_{n,g}, \vec{X}_{j,g}$ **and swap** F_j, F_n
7. **For** *n = 1* **to** *NP* **Do** steps 8, 9
8. Randomly select $r_1, r_2, r_3, \epsilon \{1,2,3,\dots,NP\}$ and $n \neq r1 \neq r2 \neq r3$
9. **Let** $\vec{V}_{n,g+1} = \vec{X}_{r1,g} + F \cdot (\vec{X}_{r2,g} - \vec{X}_{r3,g})$
10. **For** *i = 1* to *D* **Do** steps 11,12
11. Select *r* randomly $(0 \leq r < 1)$; $i_{rand} \in 1,2,\dots,D$ and Cr = 0.8
12. **If** $r \leq Cr$ ***or*** $i = i_{rand}$ **then** $\vec{U}_{i,n,g+1} = \vec{V}_{i,n,g+1}$ **else** $\vec{U}_{i,n,g+1} = \vec{X}_{i,n,g}$
13. **If** $F(\vec{U}_{n,g+1}) < F(\vec{X}_{n,g})$ **Do** step 14 **else** $\vec{X}_{n,g+1} = \vec{X}_{n,g}$; $F(\vec{X}_{n,g+1}) = F(\vec{X}_{n,g})$
14. **If** no improvement counter ≥ LS **and** $F(\vec{U}_{n,g+1}) < F(\vec{X}_{1,g})$ **then** reset no improvement counter **and do** Step 15 **else do** step 18
15. **Set** $\vec{U}_{n,g+1}$ → Local search operator ($\vec{U}new_{n,g+1}$) – Hooke and Jeeves algorithm
16. **If** $F(\vec{U}new_{n,g+1}) < F(\vec{U}_{n,g+1})$ **then do** step 17 **else do** step 18
17. $\vec{X}_{n,g+1} = \vec{U}new_{n,g+1}$; $F(\vec{X}_{n,g+1}) = F(\vec{U}new_{n,g+1})$
18. $\vec{X}_{n,g+1} = \vec{U}_{n,g+1}$; $F(\vec{X}_{n,g+1}) = F(\vec{U}_{n,g+1})$
19. **If** Stop criterion = true **then do** step 20 **else do** step 23
20. **Set** $\vec{X}_{1,g+1}$ → Local search operator ($\vec{X}new_{1,g+1}$) – H-J algorithm
21. **If** $F(\vec{X}new_{1,g+1}) < F(\vec{X}_{1,g+1})$ **then do** step 22 **else do** step 23
22. $\vec{X}_{1,g+1} = \vec{X}new_{1,g+1}$; $F(\vec{X}_{1,g+1}) = F(\vec{X}new_{1,g+1})$
23. Report $\vec{X}_{1,g+1}$ and $F(\vec{X}_{1,g+1})$

The values for the parameters of the DE algorithm and H-J algorithm will be assumed based on the recommendations made in the optimisation algorithms section.

6.2.2. Optimisation process

So far, it has been revised the approaches of pumping optimisation regard energy and costs in networks without tanks. Then, the implications of include tanks have been exposed and a new method to carry out the optimisation has been proposed. In the same way, a cost function has been formulated and each one of its elements has been explained. After that, the optimisation method to solve the problem has started to develop. In that sense, a study of the algorithms to assess the OF has been done. Thus, following the optimisation method, a list of the steps needed to reach the optimal solution is presented.

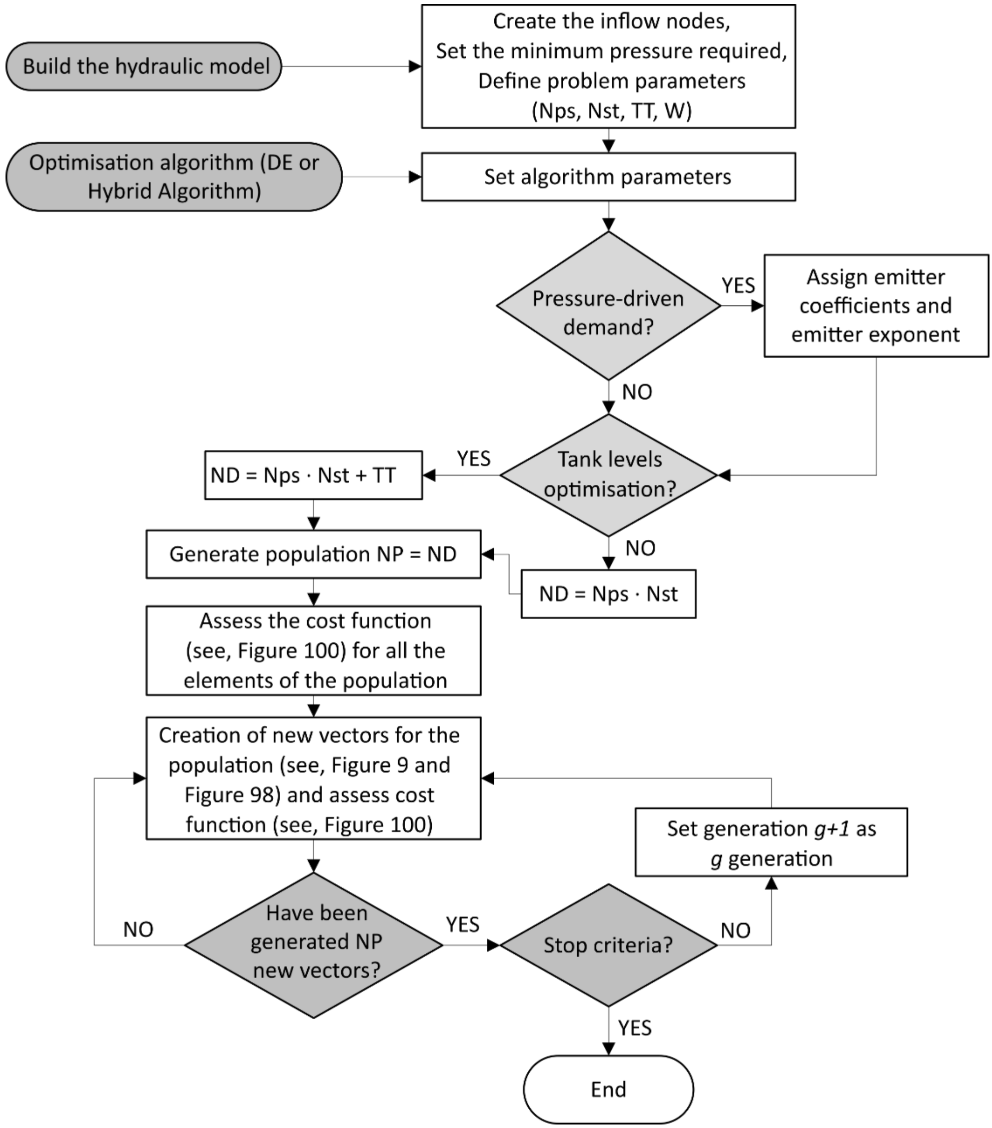

Figure 99. Energy cost optimisation by means of SCs in networks with tanks

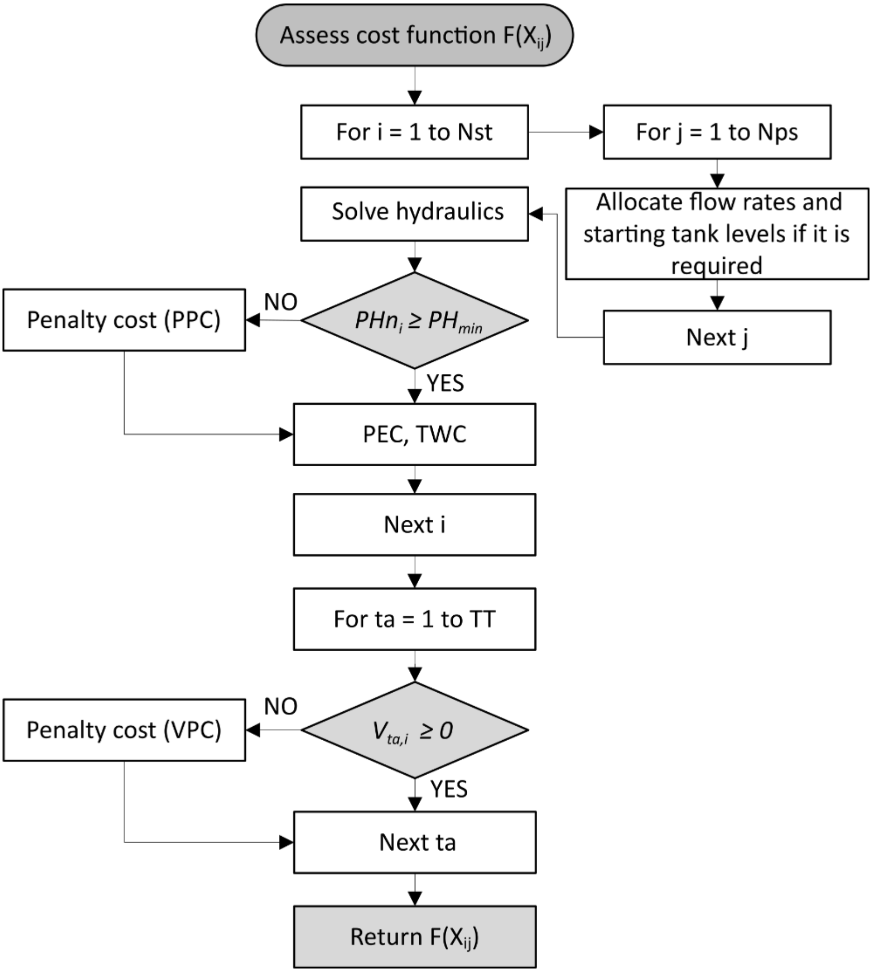

Figure 100. Evaluation of the objective function

The steps are collected schematically in Figure 99 and Figure 100 (Figure 99 shows the general method of optimisation where information related to the optimisation algorithm is placed, and Figure 100 shows only the steps followed to evaluate the OF). It could be though that the process to address the OF should be very similar to

1. The first step is initialising the parameters according to the applied algorithm. In the cases of study, it will be used the values of the next table:

Table 36. Parameters of Differential Evolution Algorithm and Hybrid Algorithm

Notation	Description of parameters	Value
	Differential Evolution Algorithm	
NP	Population number	$1 \cdot ND$
F	Weighting factor	0.5
Cr	Crossover factor	0.8
NG	Maximum number of generations	3000
	Hooke and Jeeves Algorithm	
E	Stop control value	0.01
D	Step length	0.10
	Starting point	Best value for DE
	Hybrid Algorithm	
LS	Local search activation limit	60

2. The value of the next parameters must be specified:
   - Minimum pressure required (PH_{min}).
   - Number of pumping stations *(Nps)*.
   - Total number of tanks *(TT)*.
   - Total number of stages *(Nst)* or periods of analysis.

3. The pumping stations must be represented as nodes. If there are booster pumps, they will be represented as two nodes as was pointed out before. Also factor W is defined.
4. According to the type of problem, i.e. if only flow rates are optimised ($ND = Nps \cdot Nst$) or flow rates plus initial tanks levels ($ND = Nps \cdot Nst + TT$), the number of dimensions of the problem (*ND*) will change.
5. Depending on PDD or NPDD, the emitter coefficient and the exponent coefficient should be assigned to the nodes.
6. As a part of the DE algorithm the generation g of the population is created, i.e. the *NP* elements of the population.
7. Then, the function value of each vector of the population is calculated.
8. Each time that cost function is evaluated the following steps must be followed:
   - For *i* = 1 to *Nst*
   - If level optimisation is included, set the initial levels (Equation 86)
   - Set the flow rates to the inflow nodes (Equation 84 or 85)
   - Solve the hydraulics of the network
   - Determine the pumping energy cost for period *i* (Equation 89)
   - Determine the treatment water cost for period *i* (Equation 90)

- Determine the pressure penalty cost for the period *i* (Equation 91)
- Next *i*
- Determine the volume penalty cost for the whole period of analysis (Equation 94)
- Determine the value of the cost function (Equation 88)

9. Try to improve the value of the cost function through the search algorithm criteria and create the generation *g+1*.
10. Check the stop criterion and repeat the analysis from step 7 until it is met.
11. The critical information is got: flow rates and pressure head at pumping stations, initial levels of the tanks, the minimum cost of the function, simulation time, number of iterations of the algorithm, number of generations, etc.
12. Plot the SCs and the optimal flow distribution data.

Before goes on the cases study, it should be noted that the SCs calculation process is implicit in the optimisation method. Though, this is not so evident as in the cases where tanks are not considered. The reason is that network minimum pressure and minimum storage volume must be controlled by means of adding costs to the OF function. In this context, it can be said that the optimisation process presents two kinds of critical nodes. Sometimes, the critical node will be a demand node and other times it will be a tank. The importance of each one will depend on the results of the OF. Thus, the algorithm will control the flow distribution at inflow nodes in such a way that not only the nodal pressure and storage volume but also the cost is minimum.

6.3. Cases study

With the aim of applying the developed methodology in many instances, two common networks usually implemented to evaluate different methods of optimisation will be used. These are listed below:

- Anytown network [14]
- Richmond network [15]

The Anytown network besides the pumping optimisation considers the sizing of other elements. Therefore, as the developed methodology only consider pumping cost optimisation, a previously optimised solution will be used as a starting point to apply the proposed method. Further, the original problem is subject to 3 fire flow conditions which are not included in the cost optimisation applied. Thus, the comparison with other solutions is not possible. On the other hand, the formulated approach does not consider either the selection or sizing of pumping stations. However, the method is focused on computing the optimal flow distribution and SCs for the subsequent sizing and selection of pumps. Therefore, it is plausible to get an idea of the economic benefits from the SC as the base of the optimisation process when only daily demand conditions are considered. This as long as it is possible to adapt the functioning of the existent pumping system to the SCs calculated.

6.3.1. Anytown network

Anytown network (Figure 101 and Figure 102) is well known in the design optimisation of water networks. The statement of the original problem, as well as the complete information of the network, can be found at Walski et al. [14], [64]. In general, the problem consists on selecting either new pipes, pumps, and tanks or pipes that need to be cleaned and lined. As the proposed methodology is focused on the least-cost of pumping, a previously optimised solution of the network is required as a starting point. For that purpose, the denominated "*crisp solution*" at Vamvakeridou-Lyroudia, et al. [29] has been considered. As the corresponding information of the network can be found at references, it will not be presented again, only the most relevant information will be shown in the next lines.

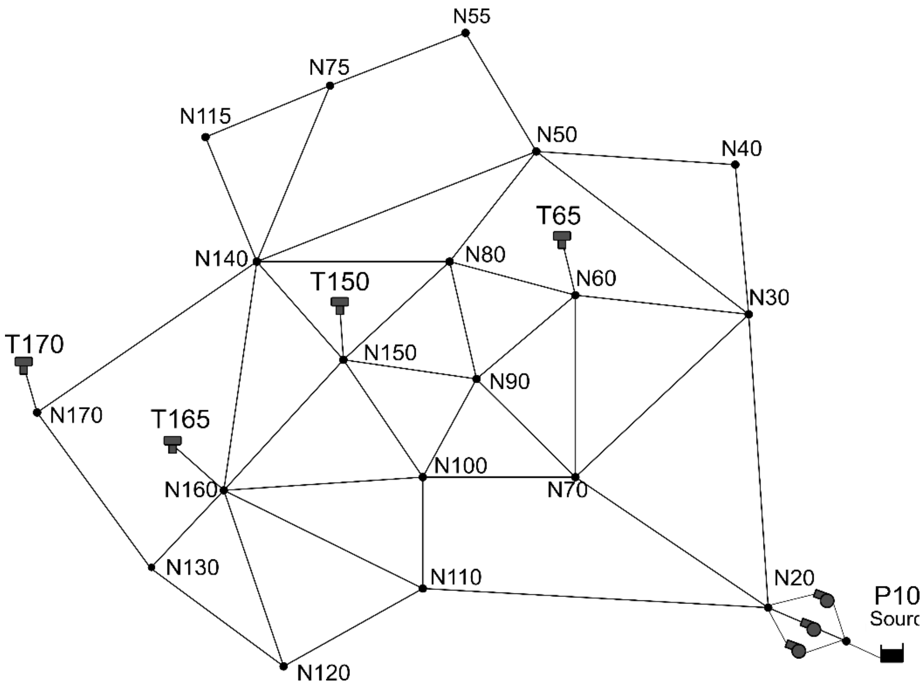

Figure 101. Starting solution Anytown network [29]

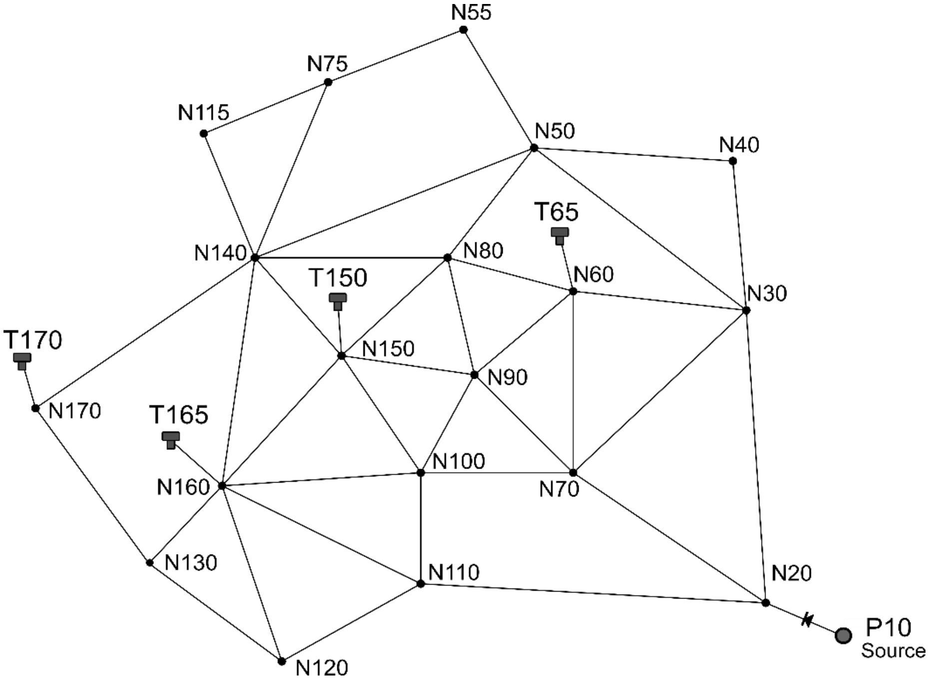

Figure 102. Anytown network prepared to apply the optimisation methodology proposed

The network has one pumping system P10 and four reservoirs T65, T150, T165, and T170 (Table 37). Minimum levels in reservoirs consider the volume of firefighting. Since the "crisp solution" fulfils all the requirements of the original problem for different loads of the system, the methodology will be applied only for the load corresponding to the daily consumptions. The minimum pressure allowed is 28.13 m. The efficiency was considered as a fixed value of 65%. This efficiency is the expected efficiency of the pumping system (Equation 89). The price of energy is a constant value 0.12 $/kWh.

Table 37. Anytown information

	Pumping station	Tanks			
	P10	**T65**	**T150**	**T165**	**T170**
Elevation (m)	3.04	68.60	70.25	68.60	66.00
Initial level (m)	-	1.41	0.28	5.55	2.56
Minimum level (m)	-	0.00	0.00	0.00	0.00
Maximum level (m)	-	7.60	4.10	7.60	4.05
Diameter (m)	-	10.89	15.94	10.89	9.85

It has been carried out a least-cost optimisation in the following cases:

a) starting with the same initial tank levels as a "crisp solution", i.e. without tank level optimisation (WOTLO),
b) making an optimisation of the initial tank levels, i.e. with tank level optimisation (WTLO),
c) changing the number of tanks considered, i.e. with tank level optimisation and just the tank 165 (WTLO_T165) and,
d) without tanks at all (WOT).

Despite the reliability is an important topic to consider when pumps or tanks are removed, it has not been contemplated. In fact, it cannot be addressed before the dimensioning of the pumping system. This because the SCs refers to the whole pumping system and the number of pumps has not been yet defined. On the other side, tanks reliability will obey the respective regulations about the minimum storage allowed. Thus, reliability is a topic to include in future works complementary to the dimensioning of the pumping system. It has to be thought that the intention of remove tanks is to demonstrate the possibility of optimising the network storage capacity as an additional benefit from the use of the SCs.

6.3.1.1. "Crisp solution"

Figure 103 shows the curves of the existent pumping system (fixed-speed-pumps) working in parallel. In the case of "crisp solution", the pumping operating points are mostly grouped over the curves of two pumps working in parallel. This solution represents a minimum pumping cost of 822,074.9 $/year. The optimisation was performed by Vamvakeridou-Lyroudia [29] using a fuzzy multi-objective algorithm. It can be noted that the optimisation was carried out by a traditional approach, i.e. by proposing a pumping system. Then, the optimal operation points were found for that arrangement. However, as the SHC of the network is unknown, it is not possible to know which is the pumping system that fits optimally with the network conditions.

Moreover, in Figure 104 the trajectory of the levels of the tanks can be observed. All of them starts with their maximum level at 6:00 A.M. hour. Besides, the initial and the final levels of the tanks are the same at the end of the simulation period. However, tanks T65 and T150 only contribute to the flow distribution 5 hours, after that they remain empty. In that sense, it looks like the storage capacity is not used efficiently. This topic will be discussed altogether with the solutions obtained after the proposed optimisation method is applied.

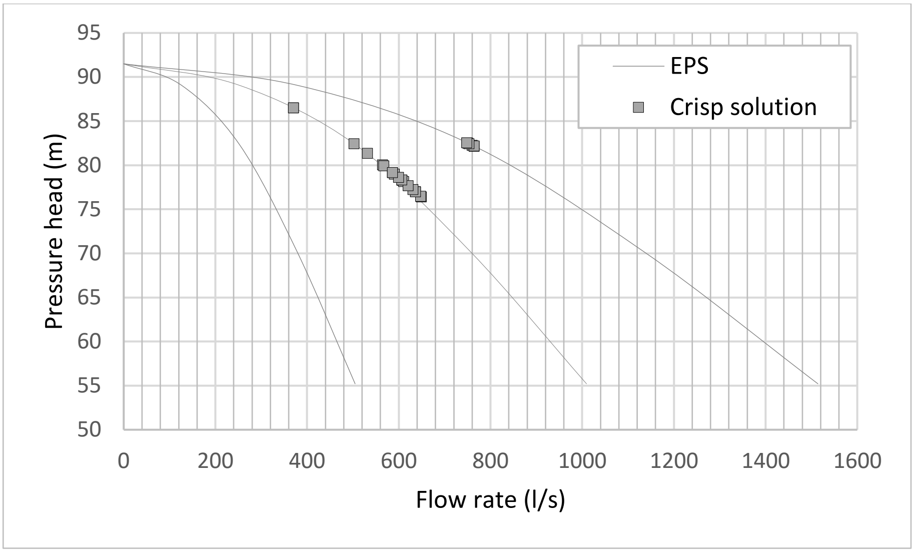

Figure 103. Crisp solution (EPS, existent pumping system)

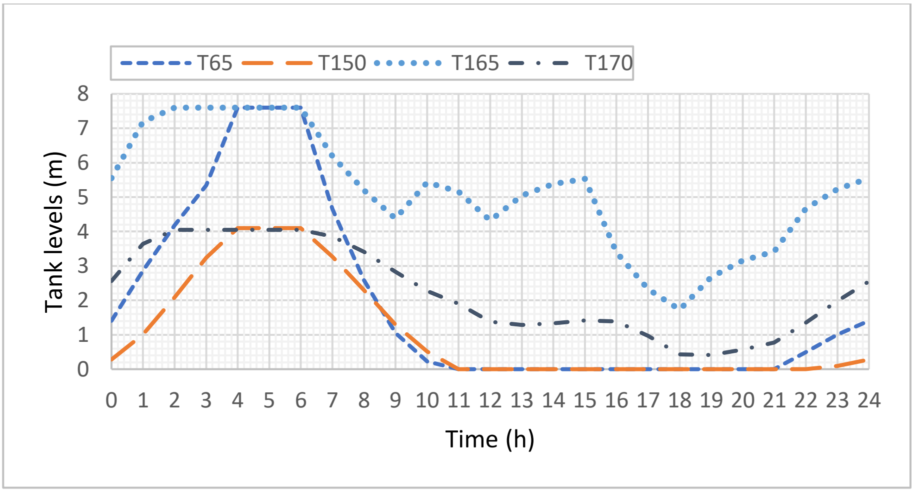

Figure 104. Evolution of tank levels starting from "crisp solution"

6.3.1.2. Optimisation without considering tank levels

Once the optimisation has been applied taking as a reference the initial tank levels of the "crisp solution", i.e. WTLO, a minimum cost of 757,552.5 $/year was achieved. This value will be taken as the point of reference to know the possible improvement of the pumping system and the tanks under the other work conditions of the network to be analysed later. The optimum SC obtained is presented in Figure 105. The SC shows the operating range of the reference solution in which there is an excess of energy of the installed pumping system. Hence, the operation cost is more considerable. Although the applied methodology leads to a minor operating costs, these can only be achieved by regulating the pumping head or by proposing a new pumping system that fits better the SC. Therefore, with the information obtained it will be easier to know the number of pumps needed at every pumping station and the type of pumps (e.g. fixed speed pumps or variable speed pumps). Also, more specific strategies can be implemented to regulate the flow rate and the pumping head in such a way that the efficiency of operation of the pumps and consequently the savings can increase.

The performance of the tank levels is shown in Figure 106. It can be noted, that despite both initial and final tank levels are the same, the evolution of tanks levels over the period of simulation are lower than the levels got from the "crisp solution". In that context, it seems is cheaper do not fill the tanks completely. This fact suggests that since the cost of the electricity rate remains constant, the use of four storage tanks could be excessive. This is because, it is more expensive to pump water to high points than to maintain the minimum pressure in the network. Storing water in the T65, T150 and T170 tanks does not represent more significant savings. Therefore, the analysis reveals the possibility of optimising the use of the storage capacity of the network. This can be translated into additional savings. On the other hand, it could affect negatively the reliability of the system, which is not considered in this study.

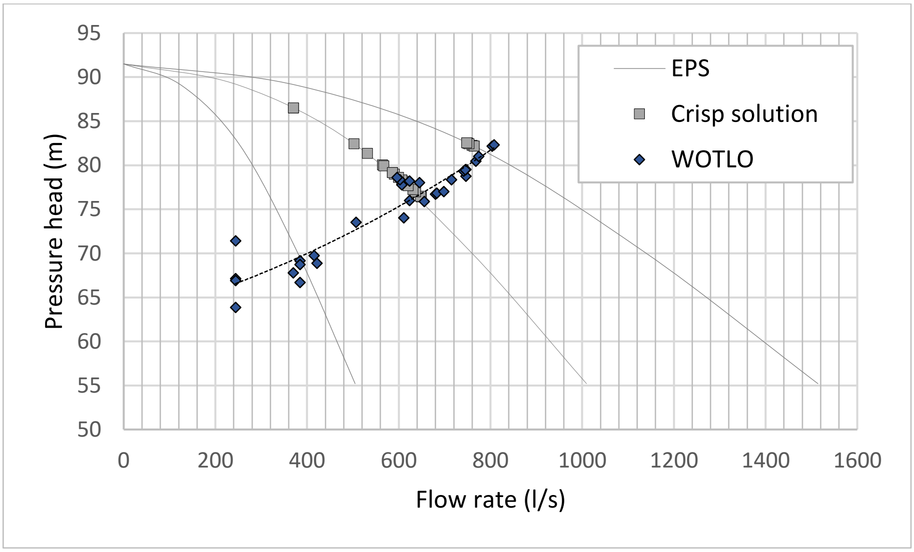

Figure 105. Setpoint curve of Anytown network without tank level optimisation (WOTLO)

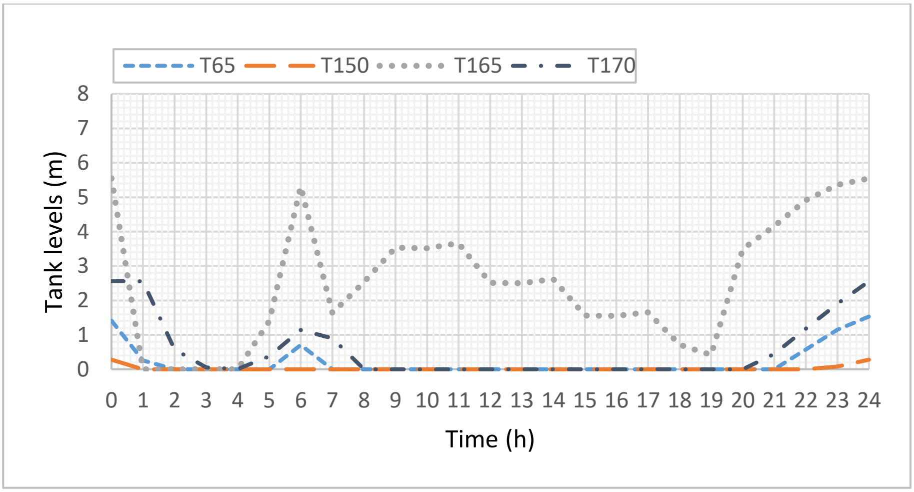

Figure 106. Evolution of tank levels without optimising them (WOTLO)

6.3.1.3. Optimisation considering tank levels

When the optimisation of the initial tanks levels is considered, a minimum cost of 747,273.8 $/year is achieved. This cost represents an improvement of 1.25 % more than in the case of WOTLO. Besides, the influence of the initial tank levels in the variations of the points of the SC can be noted. Thus, when Figure 105 and Figure 107 are compared, the points of the SC follow the tendency in a more uniform way than when tank level optimisation is included.

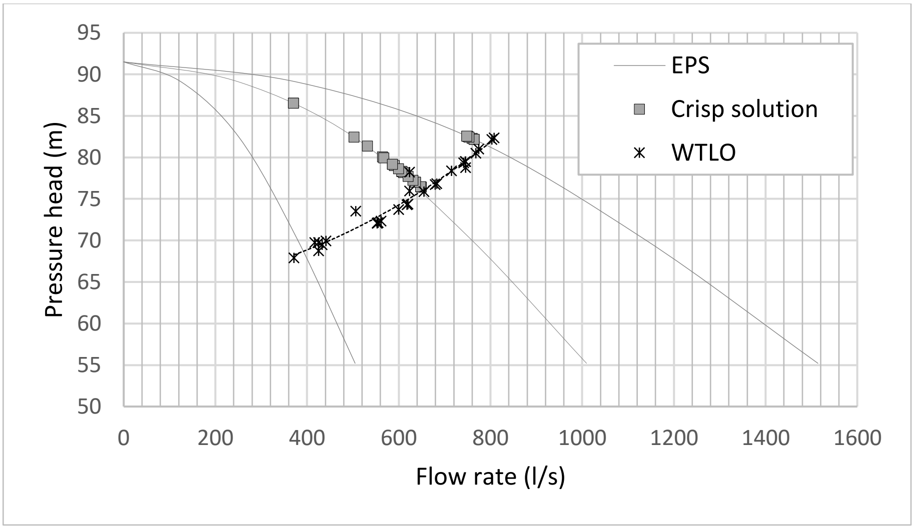

Figure 107. Setpoint curve of Anytown network with tank level optimisation (WTLO)

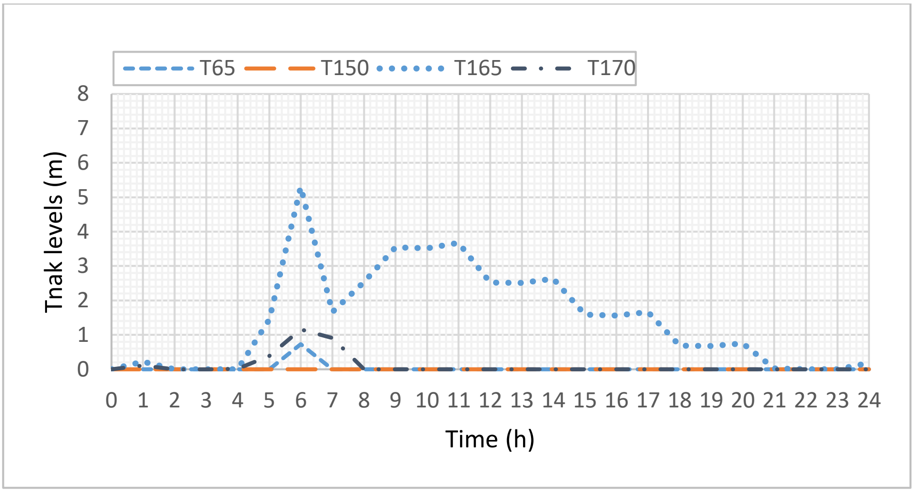

Figure 108. Evolution of tank levels considering their optimisation (WTLO)

6.3.1.4. Optimisation of tank levels considering only one tank (T165)

After performing the optimisation of the tanks levels results more evident that only tank 165 is participating in the demand cycle. Even so, the use of the storage capacity still is not efficient since the maximum level of storage is not reached. Thus, the SC concept also helps to analyse the energy and cost expenses that represent the efficient use of storage capacity in the network (Figure 108).

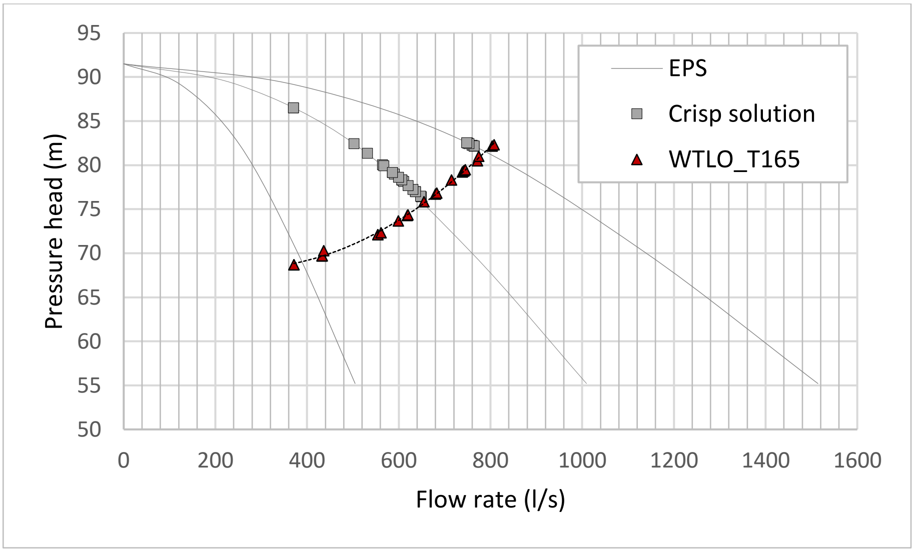

Figure 109. Optimal setpoint curve when there is just one tank (T165)

To find out what happens when only tank 165 is considered, a new optimisation has been carried out. In this case, the optimal SC is softer, and hence, the network is more balanced (Figure 109).

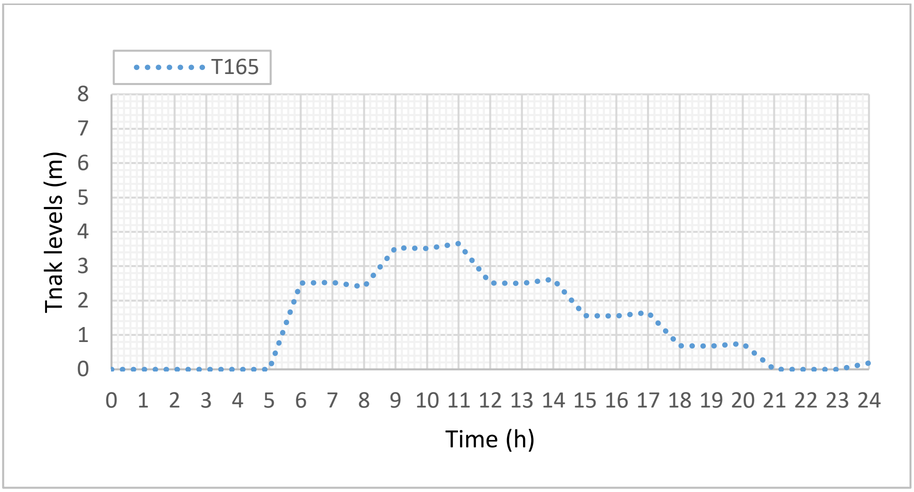

Figure 110. Tank levels evolution of Anytown network when only there is one tank (T165)

Regard to the minimum cost achieved it was 748,010.8 $/year and represents the 1.16 % of improvement against the WOTLO solution. Although it is a little worst solution than the previous solution, there is no need to use three reservoir tanks. In that sense, the savings are indirect, since less infrastructure is required to satisfy the requirements of the network. Regard the evolution of the tank levels, the use of the storage capacity is more efficient since the decrease of the storage level is more uniform over the day (Figure 110). Despite the better performance of the tank, its maximum level is 7.6 m, but only a level of 3.6 m is reached. This means, more of the 50% of the storage capacity is not used. In that sense, the last analysis will be done considering no reservoir tank available in the network trying to observe if a lower value of the function is reached.

6.3.1.5. Optimisation without tanks

Finally, if no tanks are considered the minimum cost reached is 736,157.1 $/year. The cost represents a 2.6 % more of savings against the WOTLO solution. It has to be kept in mind that as the energy fare remains constant over the whole simulation period, it is more economic pumping water directly into the system without the need of tanks at high points.

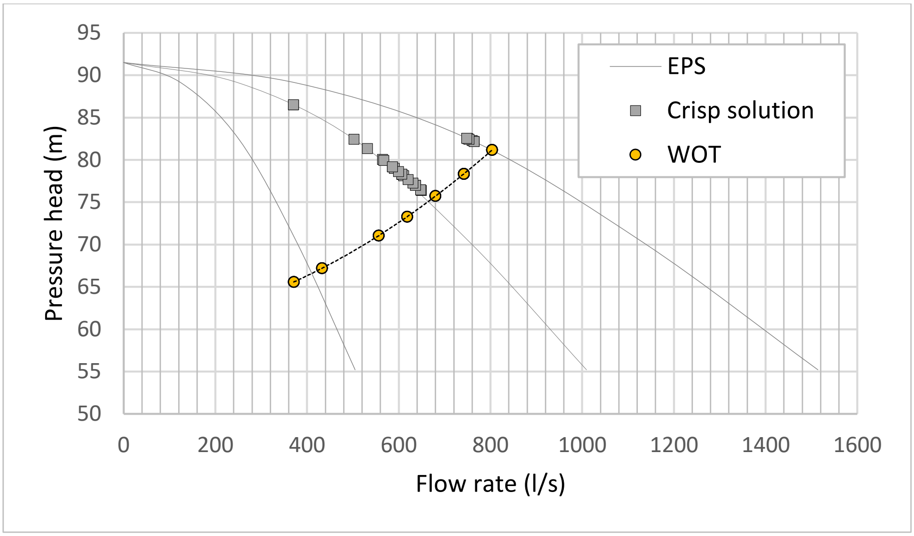

Figure 111. Optimal setpoint curve from Anytown network when there are no tanks (WOT)

To achieve the savings that result from the optimisation, it is required that the existing system adjust its operation to the SCs obtained with a minimum efficiency of 65%. Otherwise, the saving will not be as high as Table 38 shows. Since capital costs required to set the SC at the pumping stations has not been considered, for the moment, the possible savings are theoretical.

Table 38. Optimisation results of Anytown network

Description	Cost ($/year)	Saving (%)
WOTLO	757,552.500	0.00
WTLO	747,273.800	1.25
WTLO_T165	748,010.800	1.16
WOT	736,157.142	2.60

In the case that a pumping system operates with an efficiency lower than the minimum expected, good savings are still possible. This, as long as the pumping system fits the optimal SCs. To demonstrate that, an example will be developed.

For instance, a pumping system can be selected taking as a reference the optimal SC obtained when the initial levels of tanks were considered within the optimisation process (Figure 107). For that, a pumping system that fit the SC having a maximum efficiency of 65% must be selected. In the case of Figure 107, the critical point of the SC (i.e.

maximum flow rate and maximum pressure head) that the pumping system must satisfy is Q = 807.65 l/s and PH = 82.33 m. Thus, a pumping system with the same number of pumps as “crisp solution” (i.e. three pumps) has been selected. Two of them will be variable-speed pumps (VSP), and the last one will be a fixed speed pump (FSP). The two VSPs will function at the same speed all the time. All the pumps have the same size and accomplish with the Equations 72 and 73 (FSPs) and Equations 74 and 75 (VSPs). The pumping curves parameters are presented in the following table:

Table 39. Pumping curves parameters

Description	Value
a	109.7796
c	3.78671E-04
e	4.82900E-03
f	8.96800E-06

By using the proposed pumping system, it is possible to reach an operating cost of 761,923.0 $/year. It can be observed that this solution is still very close the previous solutions.

Table 40. Pumping system operating costs

t (h)	2 VSP					1 FSP			Total cost ($/h)
	Q (l/s)	H (m)	α_s (%)	η (%)	Power (kW)	Q (l/s)	η (%)	Power (kW)	
1	440.65	69.92	89.69	64.5	468.59	-	-	-	56.23
2	424.84	69.72	88.92	64.17	452.79	-	-	-	54.33
3	432.7	69.47	89.12	64.37	458.07	-	-	-	54.97
4	371.24	67.89	85.86	62.48	395.74	-	-	-	47.49
5	416.26	69.76	88.59	63.95	445.45	-	-	-	53.45
6	505.8	73.52	94.36	65	561.27	-	-	-	67.35
7	334.76	78.21	89.95	58.8	436.78	288.75	64.66	342.62	93.53
8	458.91	78.74	94.81	64.34	550.98	286.31	64.74	341.61	107.11
9	489.66	80.48	96.94	64.75	597.01	278.18	64.93	338.25	112.23
10	532.86	82.15	99.66	65	660.7	270.11	65	334.9	119.47
11	538.43	82.33	100	65	669.06	269.22	65	334.53	120.43
12	498.24	81	97.57	64.83	610.73	275.67	64.96	337.21	113.75
13	457.9	79.34	95.06	64.28	554.47	283.51	64.82	340.45	107.39

t (h)	2 VSP					1 FSP			Total cost ($/h)
	Q (l/s)	H (m)	α_s (%)	η (%)	Power (kW)	Q (l/s)	η (%)	Power (kW)	
14	462.26	79.48	95.3	64.36	560.02	282.86	64.83	340.18	108.02
15	426.7	78.36	93.32	63.52	516.36	288.06	64.68	342.34	103.04
16	383.76	76.72	90.88	61.98	466.02	295.48	64.38	345.41	97.37
17	387.74	76.82	91.07	62.15	470.16	295.01	64.4	345.21	97.85
18	355.41	75.86	89.44	60.54	436.88	299.3	64.19	346.99	94.07
19	311.78	74.28	87.21	57.66	394.03	306.16	63.78	349.84	89.26
20	314.1	74.35	87.31	57.84	396.07	305.89	63.79	349.72	89.5
21	599.51	73.7	99.06	64	677.21	-	-	-	81.27
22	552.45	72.08	95.9	64.68	603.91	-	-	-	72.47
23	554.84	72.13	96.05	64.65	607.22	-	-	-	72.87
24	561.21	72.34	96.46	64.58	616.7	-	-	-	74
							Total ($/day)		**2087.46**
							Total ($/year)		**761922.97**

In the case of the FSP, a minimum efficiency of 62.48% is achieved and in the case of VSPs a minimum efficiency of 57.66%. Hence, it can be said that the pumping system keeps operating in a high-efficiency zone. The investment costs have not been considered. However, the example is aimed to show that it is possible to obtain good efficiencies at pumps starting from the optimal SCs and therefore get closer to the maximum calculated saving. All the results that has been described (i.e. costs and efficiencies and so on) regarding the example are shown in Table 40.

6.3.2. Richmond network

The distribution model of the Richmond water network is owned by Yorkshire Water in the United Kingdom and has been used for research on some methods of optimising the pumping operation [28], [37]. This network (Figure 112) has a source of supply with a variable suction level which has an associated pumping system (1A, 2A). It is also made up of five booster pumping stations (3A, 4B, 5C, 6D, and 7F). There are six tanks (A, B, C, D, E, and F). The nomenclature of the pumps indicates the tank they are associated depending on the corresponding letter. The network data can be found on the website of the University of Exeter [15].

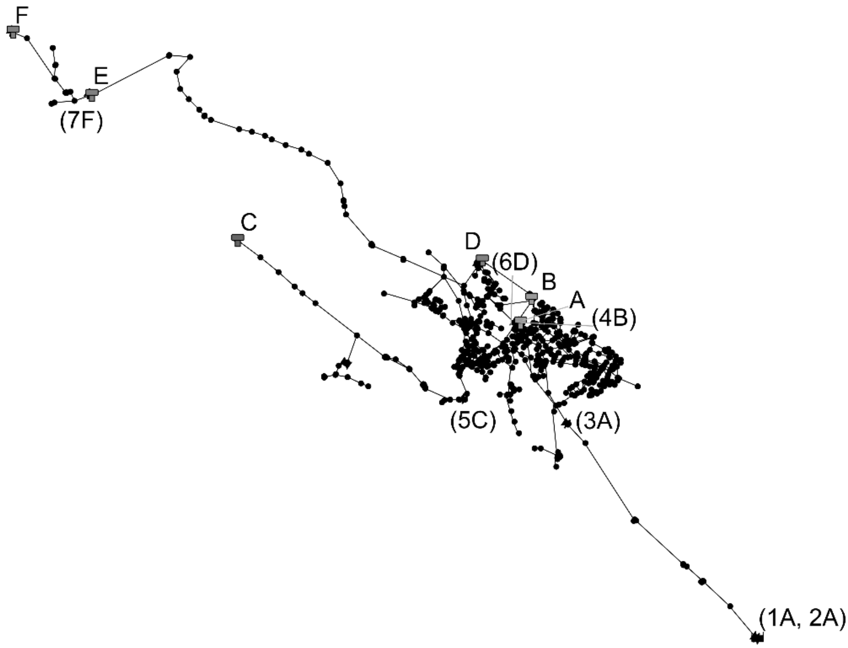

Figure 112. Richmond network

The analysis of the system is based on the assumption that the operating pressures of the demand nodes are fulfilled as long as the levels of operation of the tanks are within the pre-established ranges (in other words, minimum and maximum levels). The objective is to find the optimum levels of service of the tanks, as well as the SCs of the pumping stations that lead to the lowest energy cost for 24 hours. There are different tariffs for each pumping station, and they are divided into two phases, the off-peak (From 0:00 to 7:00) and the peak period (From 7:00 to 24:00). The data of energy tariffs and the pumping stations are presented in Table 41. The table also shows the efficiency values expected in each pumping station.

Table 41. Expected efficiency of pumping stations and energy tariffs

Pumping station	Efficiency (%)	Off Peak (£/kWh)	Peak (£/kWh)
(1A, 2A)	75	0.0241	0.0679
(3A)	77	0.0241	0.0754
(4B)	72	0.0246	0.1234
(5C)	71	0.0246	0.0987
(6D)	58	0.0246	0.1120
(7F)	54	0.0244	0.1194

Tanks information is presented in Table 42, which includes the initial levels of the tanks at zero hours (0:00 h) obtained after the optimisation process. A minimum operating cost of £ 33,982 is stated at the Centre for Water Systems of the Exeter University [15]. This cost is taken as a reference when analysing the results obtained using the methodology proposed in the present study. The cost derived from the method presented in this document is £ 29,705.96 with a saving of 12.58%. Although it is not possible to compare the different optimisation methodologies because of the conceptual use of pumping stations, this value indicates the maximum savings that can be achieved once the pump performance curves of the current installed pumping systems are adjusted to their corresponding SCs.

Table 42. Tanks information from Richmond Network

Tanks	Diameter (m)	Initial level (m)	Minimum level (m)	Maximum level (m)
A	23.5	2.050	1.02	3.37
B	15.4	2.030	2.03	3.65
C	6.6	0.500	0.50	2.00
D	11.8	1.100	1.10	2.11
E	8.0	1.992	0.20	2.69
F	3.6	1.293	0.19	2.19

In Figures 113-118 optimal SCs obtained after the optimisation are shown. Also, pump performance curves of the currently pumping system have been plotted altogether. It can be observed that in any case, the points of SCs overcome the flow rate ranges that existent pumping systems could provide. However, regarding the pumping heads, some of the pumping stations are undersized (1A-2A, 3A, 4B, 6D) and do not have enough power to meet the pressure requirements. Thus, to minimise the operating cost, install pumps that fit properly to the SCs is needed.

On the other hand, results point out that pumping stations 1A-2A and 3A (Figure 113 and Figure 114) which are installed in series supply quite similar pumping heads. That fact indicates that pumping stations are not working in series. In fact, pumping station 3A has a by-pass which pumping station 1A-2A uses to supply water directly to the network. In that sense, either the by-pass or pumping station 3A should be eliminated to improve the system efficiency.

In the case of the remaining pumping stations 5C and 7F (Figure 116 and Figure 118), although the stations have sufficient capacity to satisfy their SCs, pumping stations are oversized. Therefore, it is important to note that, since the current system is made up of fixed speed pumps, the SCs cannot be followed. Thus, the implementation of flow and head adjustment systems is necessary. This will allow that pump performance curves to work closer to the SCs. In this way, the methodology proposed allows knowing the maximum savings in operating costs that a pumping system can achieve whenever it is possible to work over its corresponding SC.

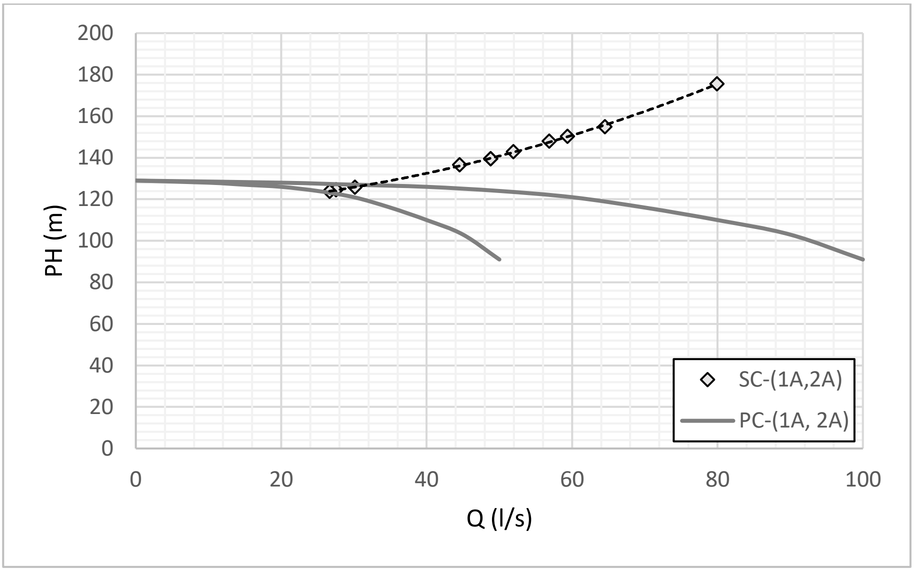

Figure 113. Setpoint curve points (SC) and pumping station curves (PC) of station 1A,2A

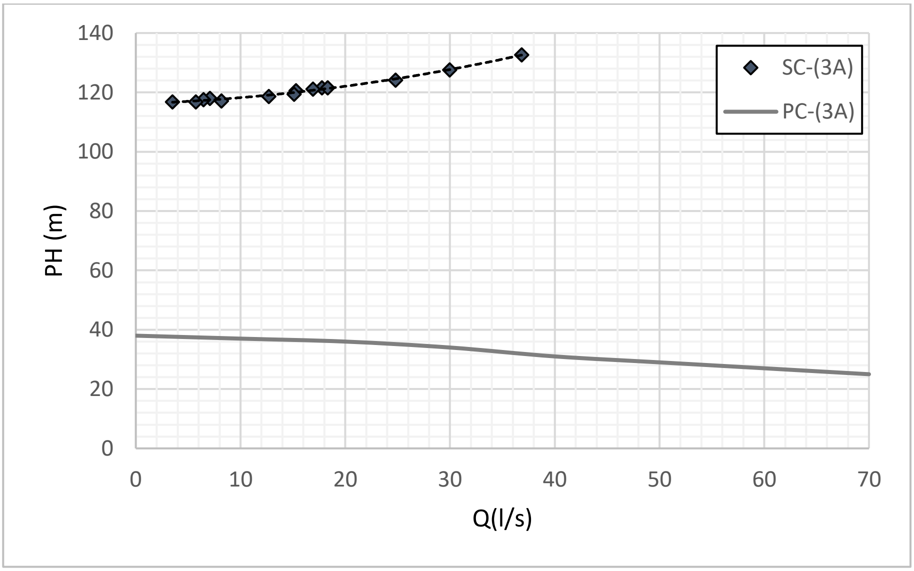

Figure 114. Setpoint curve points (SC) and pumping station curves (PC) of station 3A

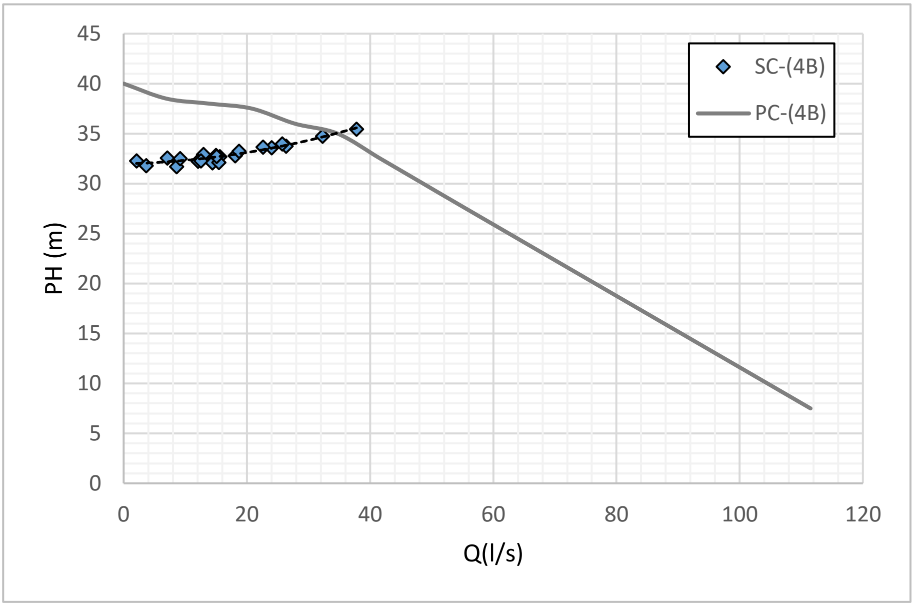

Figure 115. Setpoint curve points (SC) and pumping station curves (PC) of station 4B

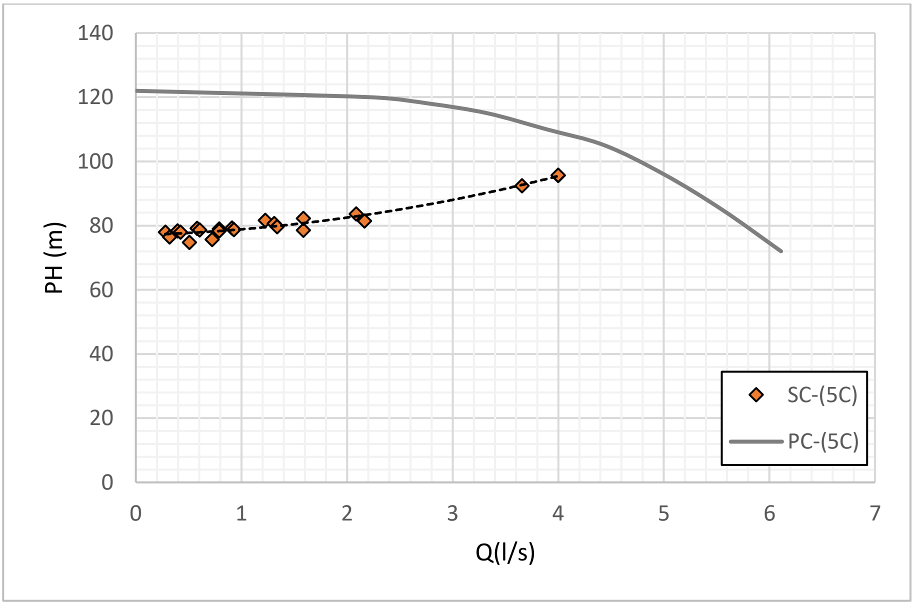

Figure 116. Setpoint curve points (SC) and pumping station curves (PC) of station 5C

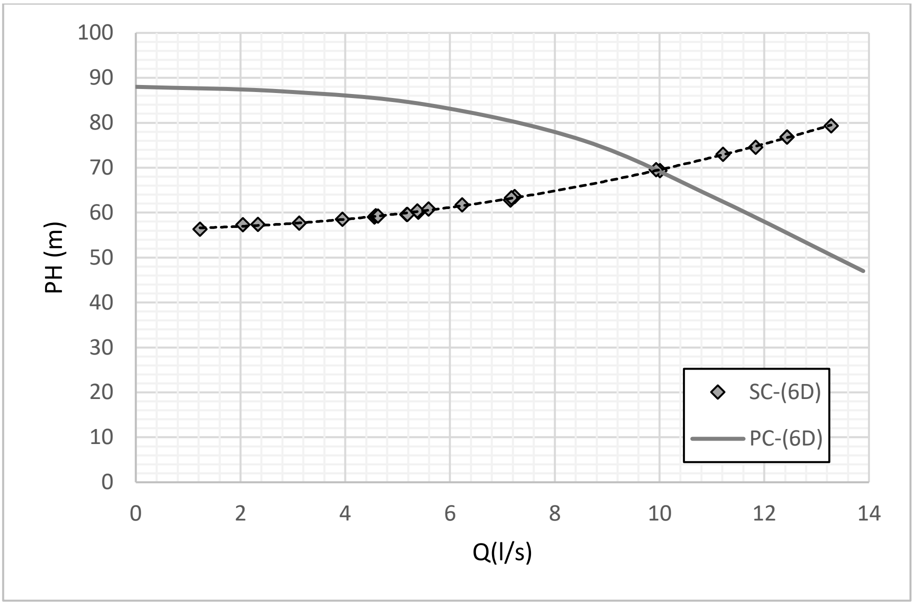

Figure 117. Setpoint curve points (SC) and pumping station curves (PC) of station 6D

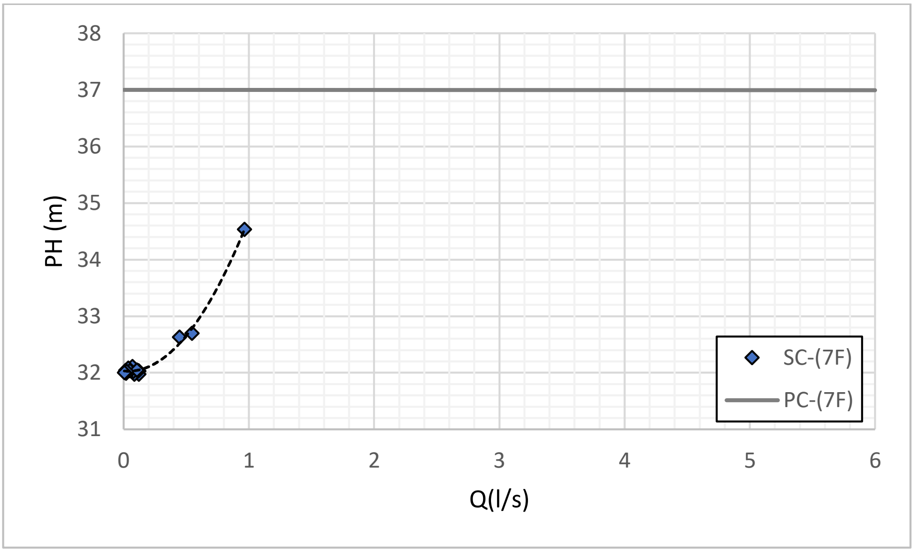

Figure 118. Setpoint curve points (SC) and pumping station curves (PC) of station 7F

For energy and cost optimisation in networks where the storage capacity is not included, the shape of the SCs varies uniformly when the demand increases. This means, that the slope between points is relatively uniform and positive. This does not happen in the case of networks with tanks, mainly because it is not possible to maintain the minimum pressure at the critical node during the entire analysis period. In fact, the pressure in this node will be higher than the minimum when it is necessary to fill the tanks of the network or when only tanks are supplying water into the network. Therefore, the SCs show oscillations subject to changes in tank levels. In that sense, the slope between points of the same SC can be negative. For instance, when the network demand increases the pressure head required by the pumping station could be lower since the pressure head at the critical node has diminished its value (i.e. tanks levels have changed and also the flow distribution) regarding the previous demand. As the pressure at the critical node varies, sometimes the SCs are formed by points with the same flow rate but with a different pressure head. Thus, the oscillations between the points of the setpoint curves make hard trying to follow them as laws of regulation at pumping stations. When this happens, it is necessary to readjust the curves to smooth the variations. In this way, it will be easier for the pumping systems to follow the SCs (Figures 119-124). To adjust the pump performance curves to the SCs, the different methods for both control and operation of pumping systems must be applied (variable speed drives, valves, bypass pipelines, flow and pressure sensors in the discharge point of the pumping station with a programmable logic controller, and many others). The readjustment of SCs will undoubtedly have a negative impact on the minimum cost of operation, in this case, the new minimum will be £ 29740.68, so the savings are reduced to 12.48%, a value that is still significant. It has to be thought that this saving only is possible if the pumping system can follow the optimal SCs.

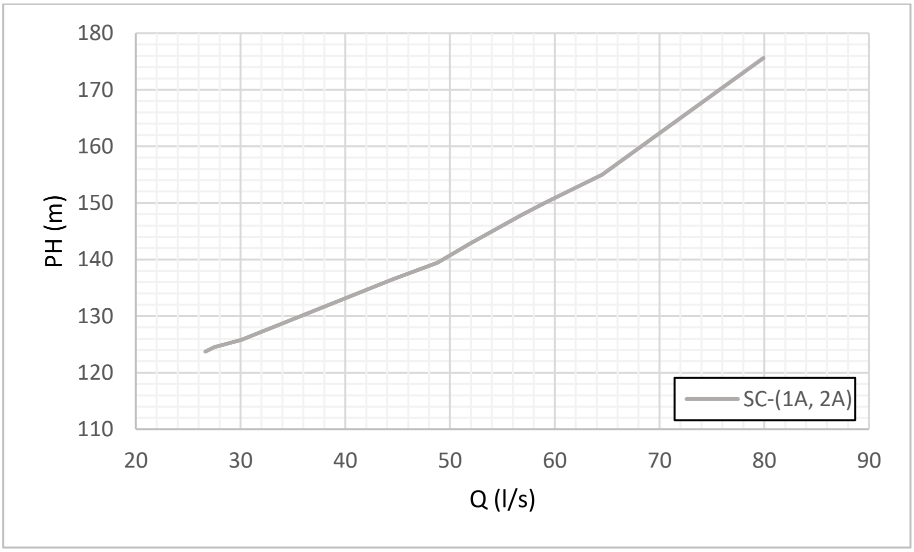

Figure 119. Smooth setpoint curve of pumping station 1A, 2A

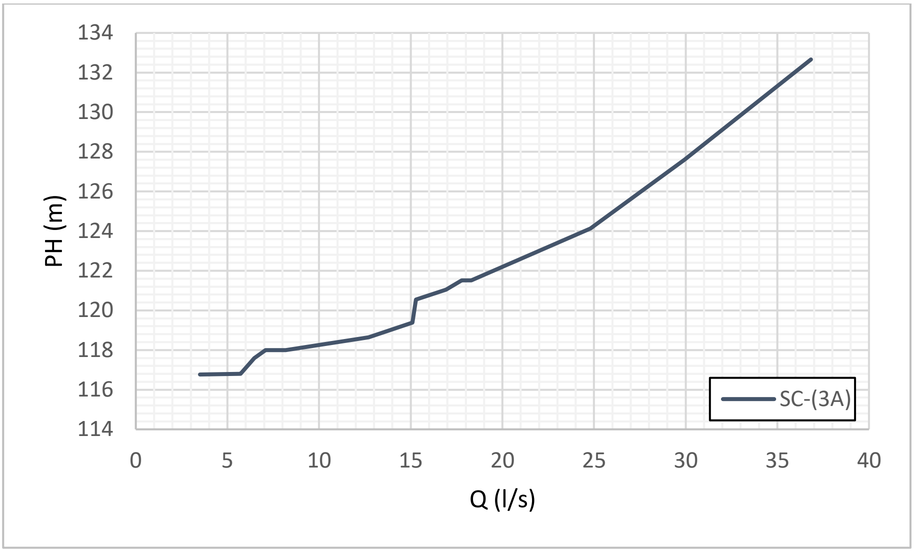

Figure 120. Smooth setpoint curve of pumping stations 3A

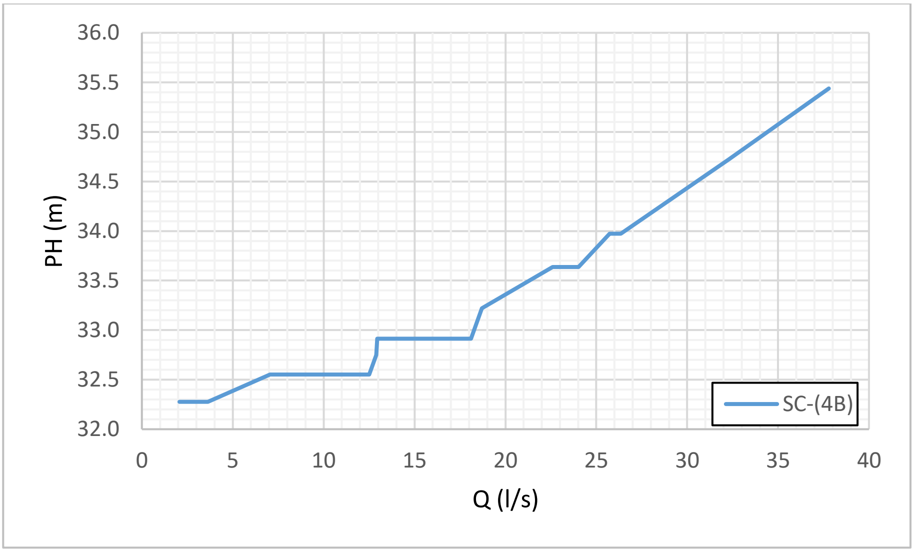

Figure 121. Smooth setpoint curve of pumping stations 4B

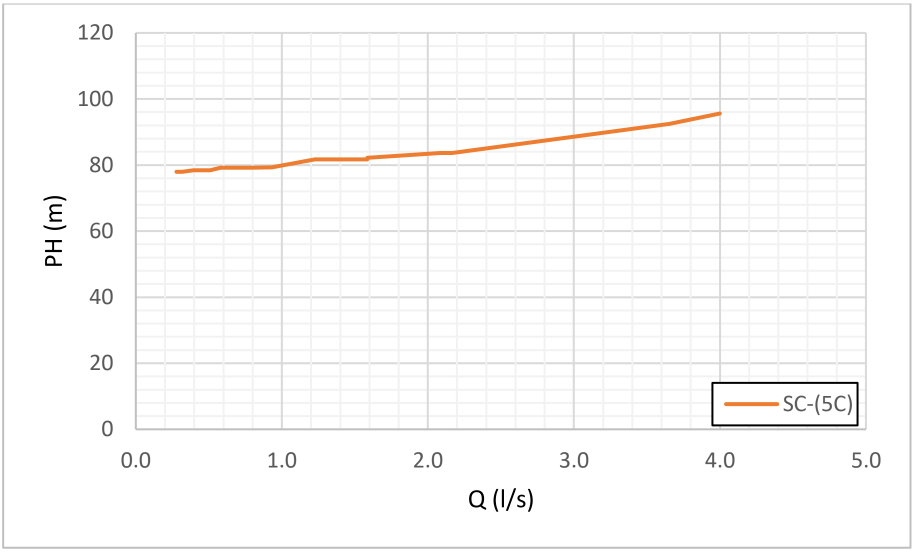

Figure 122. Smooth setpoint curve of pumping stations 5C

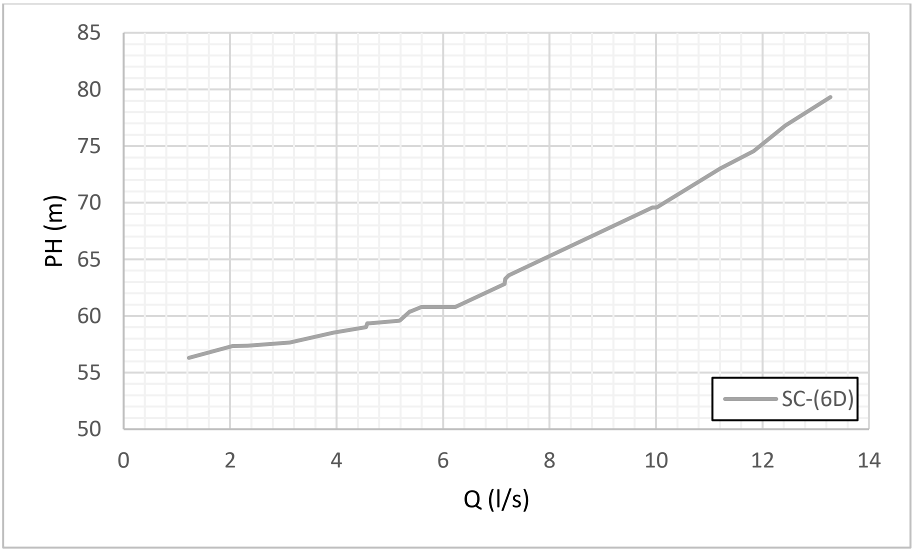

Figure 123. Smooth setpoint curve of pumping stations 6D

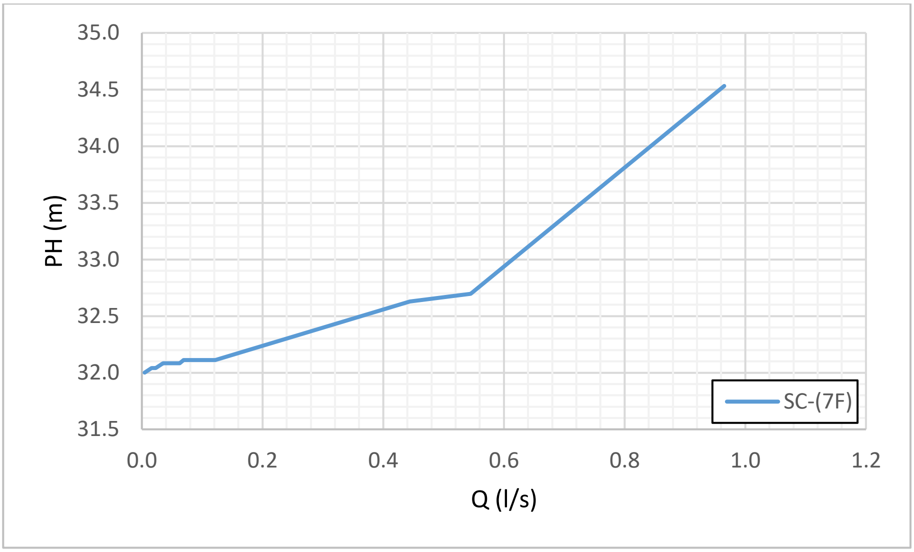

Figure 124. Smooth setpoint curve of pumping stations 7F

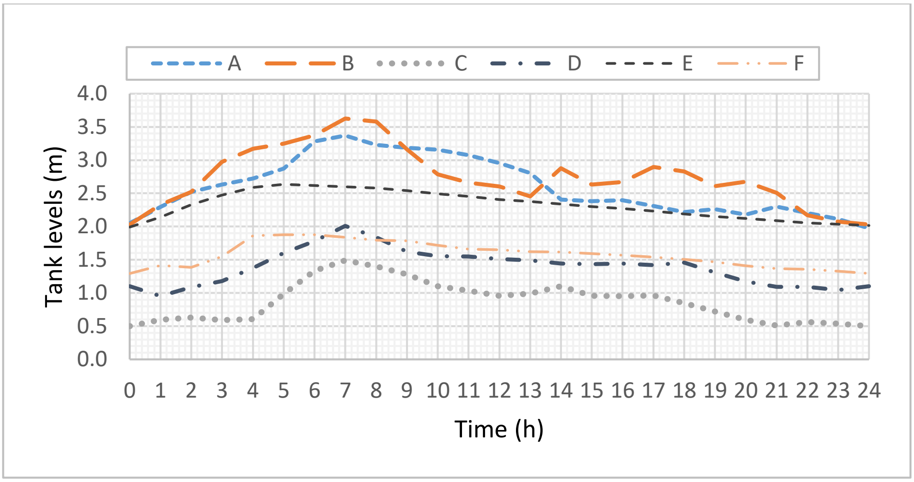

Figure 125. Tank levels evolution of Richmond

It has already been mentioned above that the concept of the SC involves keeping the minimum possible pressure at the critical node of the network while maintaining the minimum energy costs. For that, the optimal distribution of flows between the sources of supply has to be found. The use of tanks (i.e. leaving aside the reliability of the network) supposes the increase of needs of pumping energy as well as pressure head at the nodes. This energy variation could lead to a rise in operating costs depending on the proper management of energy rates. In that sense, some of the tanks do not reach their maximum available capacity over the simulation period, so that a part of the storage volume is underutilised, specifically the deposit C = 36.83% and F = 11.63% (Figure 125). In this way, the practice of pumping in hours of low energy cost until the tanks are filled and then supplying the network from the deposits in the hours of high cost proves to be insufficient. So, the optimal storage elevation which is more favourable for the cost savings must be considered. Thus, the proposed methodology also could help into the analysis of the optimal dimensioning of the tanks.

At the end of the chapter, it results beneficial to make a review of the principal statements of the energy and cost optimisation for networks with storage capacity. As the same in the previous chapter, the aim is finding the least-cost flow distribution among pumping stations for each network demand. Such distribution is obtained by applying the SC concept. However, there are some important differences to consider:

a) the hydraulic model is solved for an extended period simulation, which means a larger number of variables of decision,
b) the flow distribution among pumping stations is set depending on the average demand and a security factor (W) instead of just the total flow demand of a specific period,
c) the pressure at critical node is not adjusted by means of a dummy reservoir but by a penalty cost in the objective function, and
d) a penalty cost for non-compliance of tank levels is also contemplated.

Thus, although the problem formulation is similar than in the case of networks without tanks, the resolution is quite different. Concerning the cases study, it can be seen that important savings can be obtained in pumping operation. For that purpose, the optimal SCs must be set as laws of operation for the pumping systems. However, it has to be considered that curves are irregular due to the pressure head variation at the critical node and tank levels evolution over the simulation period. On the other side, the formulated methodology allows analysing the optimisation of the tanks of both the use and the capacity. This could be reflected in the reduction of investment costs, specifically the construction of tanks. Therefore, tanks optimisation can be thought as an adding value of the method.

Chapter 7
Conclusions

Through the presented document, the setpoint curve (SC) and its application for the energy and cost optimisation of pumping systems have been analysed. Thus, this section collects the main ideas obtained from the research. For that purpose, the setpoint calculation methodologies are taken as a starting point. Then, the different optimisation methodologies are mentioned. On the other side, the optimisation algorithms (Hooke and Jeeves, Nelder and Mead, Differential Evolution and the Hybrid Algorithm) and the cases study are treated as separate topics. A review of future developments is also done. Finally, a thesis quality indicators segment, where the different publications resulting from this work are listed, is included.

7.1. The setpoint curve calculation

For the sizing and operation of pumping stations is important to obtain the system head curves of the network. It has been stablished that there are two types of curves. The resistance curve (RC) and the SC. The RC is understood as the head-flow curve needed in the pumping station to overcome the static lift and the resistance generated by the network and the consumer to deliver the demanded flow. In that sense, the RCs are subjected to the behaviour of the user and are very difficult to estimate. Thus, it may be more convenient to find the SC instead.

The SC has been defined as the head-flow curve required in an inflow node (i.e. pumping station) to provide the minimum pressure required at a reference node of the network (i.e. the critical node) for any demand. Its calculation, leaving aside the energy and cost optimisation, has been studied for networks without storage capacity and with the following conditions:

a) one pumping station and non-pressure driven demand (NPDD),
b) one pumping station and pressure-driven demand (PDD),
c) several pumping stations and NPDD, and
d) several pumping stations and PDD.

In the two first cases, the methodology is focused on correcting the pressure head at the critical node until reaching the minimum pressure required. The adjustment is done by changing the head of the inflow node which is represented by a dummy reservoir.

In the case of networks with several pumping stations, the pressure head adjustment is performed the same as when there is only one pumping station. However, as there are more pumping stations, first a distribution of the flow to be supplied into the network among the available pumping stations must be set. Thus, when the number of pumping stations increases, the setpoint curve is obtained through two steps:

a) the allocation of the flow distribution among pumping stations to meet the demand, and
b) the correction of the pressure head at the critical node to match the minimum pressure head.

In this way, the process to compute the SC can be summarized in the two previous steps. However, it must be mentioned that, when the model includes pressure- driven demands, the steps will be repeated as many times as will be necessary to satisfy the demand and the minimum pressure head of the network.

7.2. Energy optimisation approach without storage capacity

It has been formulated a methodology to minimise the energy consumption at pumping stations through finding the optimal SC as well as the optimal flow distribution among the water sources. For that, it has been assumed that pumping stations behave as inflow nodes and each supply source has an associated pumping station.

The objective function (OF) has been built from the pressure heads and flow rates got from the optimal SCs. From there, the method to compute the SC when several pumping stations are available and consumptions depend on pressure has been studied more intensely. The SC involves the minimum pumping head that should be available to deliver a specific flow rate into the network. However, for the SC calculation process there is no need of defining any pumps, i.e. number or sizing. This because the method deals with pumping stations as inflow nodes.

It has been observed that in systems without storage capacity, SCs are different when the flow distribution among pumping stations changes. This is, when the proportion (percentage) of the demand that each pumping stations supplies to meet consumption of the network varies. However, those changes are not related to the variation of the critical node. In fact, the location of the critical node influences the gradient among the points of the SC but do not produce another curve. The variation of the SCs as result of the changes in flow distribution among the water sources do not affect either the minimum pressure kept at the critical node or the demand that has to be satisfied. But, the pressure head needs at each pumping station are modified instead. Thus, it is possible to find the optimal flow rate distribution among the water sources or pumping stations that satisfy the requirements of pressure and demand of the network with the minimum needs of pressure head. This means to minimise the energy needs of pumping stations.

When the flow is considered as a discrete variable, its optimal distribution will come from the best solution found (i.e. minimum value of the function) within a fixed set of proposed combinations of flow distributions. That is what it has been called “discrete method” (D-M) in this work. The D-M is applied in, static hydraulic models. Therefore, the OF is minimised each time that the network demand changes. Through this process, it is possible to find the energy lines that result from each flow distribution. These lines show that there is only one optimal flow distribution among pumping stations for each demand of the network. Each optimal flow distribution found represents one point of the optimal SC. Thus, through the optimisation of the flow distribution, the optimal SC of each pumping station can be calculated. Since there is only one optimal flow distribution, there is just one optimal SC for each pumping station.

The *sensitivity analysis* applied to the D-M is ruled by the two variables of the SC, the flow and the pressure head. In that context, the analysis points out that the optimal flow distribution is influenced the most for the HGL elevation at the suction node in each pumping station. Depending on the suction elevation, one source or pumping station can assume more or less percentage of the demand to be supplied into the network. This happens even for small variations of the suction level. Besides, it has to be considered that most of the time the magnitude of the pumping head is higher than flow rate values. Thus, for small variations in the network demand the optimal flow distribution tends to be constant. However, as the demand increases other factors become important, such as the diameter of the pipes, the roughness, the length of pipelines. That is, the resistance generated by the elements of the network rises, which undoubtedly ends up affecting the optimal flow distribution and makes necessary to apply a variable distribution among pumping stations.

When the number of pumping stations increases, networks become more complicated, so that the number of dimensions is higher. In that context, the disadvantage of the D-M is that the higher the number of pumping stations and the closer the optimal flow distribution to the global optimum is, the higher number of combinations to be analysed. Thus, the computing time can increase significantly, so that it has to be taken into account when the method is applied. In that sense it is more convenient to treat the flow distribution as a continuous variable, that is the continuous method (C-M). This approach lies in finding the optimal flow distribution as well as the optimal SCs by mean of a direct search algorithm. As straightforward search algorithms have problems with optimal local values, the search space is reduced by indirect restrictions, i.e. they are not included in the OF, to guarantee the optimal global solution. These restrictions can be grouped as flow distribution restrictions where the maximum and minimum flow rate that a pumping station can supply is limited.

Independently of the method discrete or continuous the process to calculate the SC implicit in the optimisation process, guarantees that the minimum pressure always is kept at the critical node. Thus, the OF only analyses the proposed flow rate distribution combinations, as well as pressure heads, that have been obtained from the SCs, to reach the optimal solution.

7.3. Cost optimisation approach without storage capacity

From the C-M, this research stage considers pumping and treatment cost optimisation. This process is required to determine the least-cost utilisation of multiple sources from which the water is pumped into a distribution system. For that purpose, the optimum flow rate must be found for each of the sources/pumping stations over the period of analysis (e.g., 24 hours). As there are no tanks, the hydraulic model is solved for static state conditions and costs are minimised for each network demand separately.

In addition to the energy consumption, the OF developed in this section allows consideration of additional aspects, such as water production costs, electricity tariffs, and the minimum and maximum flow rates for the sources. It is also possible to add any other consideration relevant for the particular network, e.g., water quality, as long as it can be expressed as a cost. The assessment of the OF requires the use of an optimisation algorithm. The algorithm must allow exploration of the wide range of water supply combinations among the water sources associated with pumping stations. In this case, it happens the same as in the energy approach (i.e. the hydraulic model is static, and flow distribution is constrained indirectly). Hence a direct search algorithm is efficient enough to deal with the problem. Through the incorporation of the costs in the OF not only the least-cost flow distribution is achieved, but the least-cost SCs.

The cost optimisation starts from the assumption that the pumping system that fits better with the features of the network is unknown. Further, the SCs does not need that the pumping system is dimensioned previously (i.e. number of pumps and pump performance curves). Thus, to calculate the pumping costs, the OF considers a constant parameter defined as expected efficiency. This parameter is assumed as the minimum efficiency expected of the pumping system. Therefore, when the efficiency is the minimum expected, the minimum saving possible under the conditions established is found. This way, when pumps are selected following the optimal SCs calculated, a higher saving could be reached.

7.4. Energy and cost optimisation with storage capacity

In the third approach of the research, a methodology that allows finding out the optimal pumping points to reach the least operating possible cost in networks with storage capacity has been presented. Although the method is based on the SC as first and second sections did, the approach is different.

A fundamental part of the methodology is to represent all the pumping stations as inflow nodes. Regarding booster pumping stations two nodes will be used, the first one will be the suction node of the pumping stations and the second will be the discharge node. Unlike the other two approaches of this research, there is not a node of head type (i.e. dummy reservoir) to adjust the minimum pressure at the critical node. The reservoir is not needed due is not possible to keep constant the minimum pressure at critical over the whole simulation as happens in the other approaches. One reason is that when tanks are filling the average pressure in the network is higher since tanks are at higher elevations than consumption nodes. On the other side, when only tanks supply water and pumps are not working the pressure at the critical node cannot be adjusted by mean of pumping stations. Thus, there is no sense in creating a dummy reservoir to set the minimum pressure head at the critical node as was done in the other parts of the research. Besides, it must be pointed out that in both the energy and cost optimisation in networks without storage capacity the causes of variations of SCs were the change in flow distribution among water sources or pumping stations and adaptation of the critical node. However, when storage capacity is considered, SCs also changes according to the difference of the tank levels.

The base of the method is to find the optimal flow distribution among pumping stations that minimise the cost function accomplishing three criteria:

a) keeping the minimum possible pressure head at the critical node according to the network requirements,
b) meeting the demand, and
c) comply the storage constraints.

For that purpose, it is assumed that:

a) pressure heads at consumption nodes are reached as long as the minimum pressure at critical node does,
b) pumping stations are not defined, so that the sizing of pumps as well as the number of pumps are not known, and
c) the storage tanks are located at points high enough to guarantee the minimum pressure head at critical node.

The hydraulic model is analysed in the extended period. This mean, it is a quasi-static type**.** The input information to develop the optimisation involves the flow rate discharge of each pumping station over the whole period of simulation and/or the initial tank levels (depending on if levels optimisation is carried out). The output data are the SC points of each pumping station and tanks levels variation over the period of simulation. Although the approach is not limited by the characteristics of installed pumping systems that there could be in the network (e.g. flow rates, pumping heads, pumps number), those limitations can be included if it is necessary. It has to be highlighted that since the optimisation process assumes the pumping stations as inflow nodes, there is no need to consider the pumps one by one. Hence the number of state variables (e.g. flow rates, pumping heads) is lower.

The OF is formed by four costs:

a) the pump energy cost (PEC),
b) the treatment water cost (TWC),
c) penalty cost for non-compliance with minimum pressures (PPC), and
d) penalty cost due to non-compliance of water volume storage restrictions (VPC).

The *pumping energy costs* take into account flow rates and pressure heads got from the SCs, energy fares with hourly variations, the pumping time, and the parameter of expected efficiency which allows transforming in costs the points of the SCs. The *treatment water costs* involve the energy operating costs to treat the water (e.g. chemicals, additional pumping costs, among others) through a fixed fare of treatment per flow handled. The last two costs refer to penalty costs that influence the cost function directly. The *penalty cost of pressure* head is added to the OF always that a node has a pressure head under the minimum required. The *penalty cost of the volume* is added at the end of the period of simulation each time that inflow volume of water of a tank is not the same that the output volume. Additional network operating features and constraints can be included as long as they are expressed in costs. Thus, the optimisation can be made with a unique OF.

Due to the significant number of variables (i.e. flow rate distribution for each pumping station over the simulation period and starting tank levels) and since the search space is not directly constrained within the SC calculation process, it is not possible to apply direct search algorithms. In that sense overcome problems with optimal local values is imperative. Thus, evolutive algorithms have been required to guarantee the optimal global solution.

7.5. Optimisation algorithms applied

The algorithms selection responses to a non-linear, non-derivable multidimensional OFs with restrictions. The restrictions are given depending on the case of optimisation by pressure heads, flow distribution, or tank levels.

Both in energy and cost optimisation two direct search algorithms have been applied: Hooke and Jeeves (H-J), and Nelder and Mead (N-M). In both cases the restrictions are indirect (i.e. constraints are implicit in the SC calculation), so that search space is limited indirectly. Although both algorithms have problems with local optimal solutions, the search space limited by constraints makes them efficient enough to find the global minimum value of the function within the specified area.

Since the two algorithms have reached the same results, only H-J algorithm outcomes have been presented in the document. On the other side, it could be said that if both algorithms are contrasted, H-J algorithm is more comfortable to program and faster when the minimum is reached. Though N-M algorithm is more reliable because it is based on the population information to reach its optimal solution. The parameters of both algorithms have been set observing that calculations tasks do not be excessively time-consuming.

In the case of the energy cost optimisation in networks with storage capacity, the hydraulic model is dynamic (quasi-static), the number of dimensions is higher, and restrictions are not implicit (i.e. the SC calculation process is different). Hence, they are directly included in the cost function. In that sense, the risk to be trapped in optimum local values is major and an evolutive algorithm must be applied. In this case, the Differential Evolution algorithm (DE) has been used. Although the algorithm does not always yield the same results, these are always quite close, so it can be said that the algorithm is sufficiently efficient to find the global optimum.

It has been observed that the algorithm is quite fast when discarding the non-feasible search areas. However, there are times when the function becomes stagnant and many iterations must pass before the OF improves significantly. This excessive number of iterations is translated into a time-consuming task. Besides, it has been noted that whenever a direct search method is applied to the best solution obtained by the DE algorithm, most of the time it is possible to improve it in some way. Thus, aiming to minimise the time of computation of the DE algorithm, a Hybrid Algorithm has been proposed. The main idea of the algorithm is improving the elements of the population by mean of a direct search algorithm, in this case, H-J algorithm. Thus, the local search is activated when two conditions are accomplished:

a) the algorithm has reached a prespecified number of iterations, and
b) a new minimum value of the function better than the current best value has been found.

In that context, the hybrid algorithm does not pretend to activate the local search to improve only the best value when the global search is not able to improve the function but to improve each new best value. Although more experimentation is required with the application of the algorithm, it has been observed that the time of computation has decreased at least twice that when the Differential algorithm is only used. Although four algorithms have been implemented, it has to be highlighted that the aim of the research lies in testing the different optimisation methods and not the optimisation algorithms. However, it has been tested its efficiency to achieve the expected optimum results.

7.6. Cases study

In total five networks have been implemented to assess the application of the methodologies developed: TF, Catinen, COPLACA, Anytown and Richmond. Depending on the optimisation approach the operation conditions tested change.

7.6.1. Energy optimisation without storage capacity

In this case, three networks have been tested:

a) TF network with two and four pumping stations for PDD and NPDD,
b) Catinent network with three pumping stations for PDD and NPDD, and
c) COPLACA network with seven pumping stations, PDD and flow rate limitations at pumping stations.

Through the D-M, energy lines that result from trying different combinations of flow distributions among pumping stations were obtained (see, section 4.5.1.). These lines show that there is just one optimal flow distribution that leads to the minimum energy consumption. Besides, the optimal flow distribution changes as the demand of the network vary. Finally, when the optimal flow distribution is calculated also are optimal SCs which converge in only one optimal SC for each pumping station. The case studies provide an improved understanding of the field of application of the methodology exposed. Essential questions can be answered, for example, those related to the identification of critical water sources, the influence of its location, the quantity of water to be provided by each source, and so on. As more complex networks are used (e.g. TF with four pumping stations, COPLACA) and consumptions dependent of pressure are considered the D-M seems to be not accurate enough to reach the optimal distribution and needs more time of computation. Therefore, the use of the C-M is preferable, though the energy curves cannot be got by mean this method. On the other side, another significant result is the definition of the shape of SCs and the flow rate ranges of each pumping stations according to the different operating conditions of the networks. For instance, in the case of COPLACA network, the optimal SCs look more like a line than a curve.

7.6.2. Cost optimisation without storage capacity

In this case, two networks were analysed:

a) TF network with three pumping stations and PDD, and
b) COPLACA network with seven pumping stations, PDD and flow rate limitations at pumping stations. This time, energy tariffs, unit treatment costs, expected efficiencies and demand curves were considered in the analysis.

The problem of determining the optimum flow and pressure heads for each source/pumping station in a water distribution system is complicated due to the non-linear nature of network behaviour, its topology, pressure dependent demands, and variable tariffs. Therefore, it is difficult to infer optimal pumping operating policies without a formal optimisation approach.

For TF network, a pumps selection has been done. In that sense, it has been proved that is possible to find a pumping system that fits with the optimal SCs obtained. However, the optimal pumping system has not been found. That is, a different number of pumps, size, operation, and other alternatives (use of hydropneumatic drums, valves, etc.) may lead to better and cheaper solutions. Thus, pumps sizing and selection by using the SCs is a much more complex problem that deserves a much more comprehensive study. Nevertheless, the solution to this problem is out of the limits of this work.

If results from COPLACA network are compared with those from the energy optimisation approach, it can be seen the influence of costs regards:

a) the number of pumping stations available,
b) the variations in the optimal flow distribution besides the calculation of the optimal SCs,
c) the shape of SCs and the range of flow rates within which the pumping stations operate.

This information will be undoubtedly useful in the sizing, and operation regulation of pumps. Also, through the proposed optimisation methods, it is feasible to know the importance of each of the sources of supply regarding energy and costs. This aspect may be useful when carrying out economic studies on the optimisation of the operation of the plants and the increase or reduction of their treatment capacity.

7.6.3. Energy and cost optimisation with storage capacity

For the case of networks with storage capacity two benchmark networks have been tested:

a) Anytown network with one pumping station and four storage tanks, and
b) Richmond network with one central pumping station and five booster pumping stations as well as six storage deposits.

Anytown network has been analysed under the following operating conditions:

a) without optimising the starting tanks levels,
b) optimising the starting tank levels,
c) changing the number of available tanks, and
d) without any tanks available.

The original problem considers multiple operating conditions which have not been studied. Therefore, the following comparisons are done only to give an idea of the possible savings if the existing pumping system had variable speed pumps and the average efficiency were 65% (maximum efficiency of the currently installed pumping system) or higher. In that context, results show that a minimum cost of $ 747,273.8 can be achieved by optimising the current operating conditions of the network, i.e. tank levels and pumps operation points. This is as long as pump performance curves of the existing system can work over the optimal SC points. Though, lower efficiencies may lead to lower savings but equal significant. On the other hand, if a pumping system with better efficiency values is selected (i.e. more than 65%), then the savings will be higher than the values estimated. However, in case that pumps need to be replaced, the savings must be re-evaluated because of the capital costs which are not part of this study.

After applying the optimisation method, it was possible to notice that the storage capacity was underutilised. Since the energy tariff is constant over the whole period of simulation, the criterion of pumping in low-cost energy hours and use the tanks in peak cost hours is not applicable. Thus, it is more expensive to pump water to high points than to maintain the minimum pressure in the network at the critical node. In that sense, all tanks were removed, and a minimum cost of $ 736,157.142 was reached. This value represents an improvement of 2.6% against the other solutions. In this context, the optimisation by means of the SC evidence additional applications as a tool for the optimisation of the tanks usage.

In the case of the Richmond network, a more complex network is analysed, i.e. with several pumping stations, different efficiencies, different energy fares, and several storage tanks. Results show that a maximum theoretical saving of 12.58% is possible. Besides, optimal SCs show those pump systems that are both oversized or undersized. On the other side, the methodology permits to know and optimise the interaction among the pumping stations as it happens between pumping stations 1A-2A and 3A, where results point out that pumping station 3A does not represent any saving as a booster pumping station.

Even though SC shows oscillations subject to changes in deposit levels, which make them difficult to follow by pumping stations, the variations can be smoothed, and savings can be still high. Regarding the storage tanks, their infrastructure still being underutilised, this despite the difference of the electric tariffs. In that sense, the practice of pumping in hours of low energy cost until the tanks are full and then supplying the network from the tanks in the hours of high cost proves to be insufficient. Thus, find out the optimal storage elevation which is more favourable for the cost savings must be considered. Therefore, the proposed methodology also could help into the analysis of the optimal dimensioning of the tanks.

7.7. Future developments

In this work, least-cost setpoint curves are obtained. The SCs are a type of system head curves (SHCs). Therefore, SCs application is directly related to the sizing of pumping systems. In that sense, this research does not address either the selection of fixed nor variable speed pumps that fit better with the optimal calculated SCs. Moreover, based on the computed SCs, problems like the optimal number of pumps, their optimal operation, the optimal efficiency work area and operation control methods (i.e. variable speed drives, control valves, and others), can be studied. For that purpose, further research must be done. Besides, these problems can be addressed separately with the help of the optimal SCs, i.e. without the need to solve the network. In this way, an important advantage is obtained by reducing the time and computing resources that are needed.

When there are tanks in a network, if they are located too high it could be expensive to fill them. On the contrary, if they are placed in a too low elevation, they will be another consumption node without real energy saving. Since the SC is used to find the minimum energy needed at pumping stations to satisfy the pressure requirements of the network, it can also be used to find the optimal location and sizing of the tanks. In such way, tanks will contribute to the energy and cost optimisation in a water distribution network by reducing the head requirements of the SCs.

The SC guarantees to keep the minimum pressure over the whole simulation period in networks without tanks. In the case of networks with tanks, the pressure is the minimum pressure possible (i.e. the minimum or a little bit above). In this context, the SC could be applied as a strategy for leakage control and cost reduction of the urban water management. In that sense, by estimating the leakages cost it could be known how much more savings can be achieved by setting the SCs at pumping stations. Besides, it may be interesting to combine the leakage control with the optimal flow distribution process presented in this research. That is, to estimate the costs of leakages due to the pressure of the network and to contrast them with the operating costs of the pumping. This added cost could affect the optimal flow distribution more than the energy costs as long as the leakages cost becomes more relevant.

It must be kept in mind that the optimisation methodology presented in this work is based on finding the minimum energy (in terms of head) and associated costs, required at water supply sources. For that, a specific pressure head is set at the critical node. Thus, this concept could be applied, combined with multi-objective functions, to obtain cheap network designs via genetic algorithms. This process could be done by finding the optimal location of the critical node and designing the network based on it. On the other side, the application of zoning strategies and management measures to improve the performance of the critical node have not been yet addressed.

Since the point of view of the network demand, multiple operational conditions have not been analysed. This could result in multiple optimal SCs in the case of networks with several flow demand curves and different pressure requirements. Also, in fire flow conditions likely additional operating points will be obtained. These aspects will affect the pumps selection and must be taken into account. Thus, further research is needed

Recalling the cases study, it has been mentioned that the computed savings only are possible when the existent pumps can operate over the optimal SCs or when the pumping system is designed from scratch. However, when new pumps are required instead of the installed ones or maybe some kind of equipment to regulate the operation of the pumps, new costs must be added to the OF. These are the capital costs and have to be considered to find the real savings. Besides, it has to be thought that the problem is not limited only to determine the inversion costs. Actually, capital costs will be affected by maintenance costs, the optimal number of pumps, the optimal operation, the optimal methods of operation, among others. These are aspects that also have to be studied.

In a complementary way, the reliability is another important topic to consider when some pumps or tanks are removed. However, it can be addressed only when pumping system has been totally sized. This because the proposed optimisation methodology is designed to find the optimal flow distribution considering a variable number of pumping stations. This means the flow and pressure requirements of the network will be satisfied even with just one pumping station. Besides SCs do not refer to a specific number of pumps but to the pumping system characterised by their respective curves. In that sense, no analysis can be done in regard to the number of pumps that are operating in a specific time or stop working. In the case of tanks elimination, the reliability will be given by the respective regulations of minimum storage volumes allowed. Thus, the reliability of the network must be analysed as a complementary work to the pumping system dimensioning.

From what has been mentioned above, it can be concluded that the present research opens the path to many challenging works to will be developed further on.

7.8. Quality indicators

Over the advance of the present research, most of the ideas presented in this document have been exposed in different conferences and environments, in both languages Spanish and English. They are listed in the next table:

Table 43. Conferences where the ideas of the research have been exposed

Conference	Place	Title
IV Jornadas de Ingeniería de Agua (JIA 2015)	Cordoba, Spain	Optimización del reparto de caudales de suministro en redes de distribución de agua con múltiples sistemas de bombeo
XXVII Congreso Latinoamericano de Hidráulica	Lima, Peru	Caudales óptimos inyectados en redes de distribución de agua malladas con múltiples fuentes de abastecimiento en régimen por bombeo manteniendo el mínimo consumo energético y costos
20 TH International Congress on Project Management and Engineering	Cartagena, Spain	Optimización energética de los caudales de suministro de una red de distribución de agua con múltiples fuentes de bombeo
14 TH CCWI Computer and control for the Water Industry	Amsterdam, Holland	Cost optimization of distribution looped networks through determination of optimal pumping flow rates of each of their supply sources based on the setpoint curve concept.
WDSA 2016 Water Distribution System Analysis	Cartagena, Colombia	Energy optimization of supplied flows from multiple pumping stations in water distributions networks
V Jornadas de Ingeniería del Agua (JIA 2017)	A Coruña, Spain	Optimización de costos de bombeo en redes de distribución de agua con capacidad de almacenamiento mediante el uso del concepto de curva de consigna
15TH International Computing and Control for the Water Industry Conference	The Diamond, University of Sheffield, UK	Pumping Cost Optimization in Looped Water Networks with Storage Capacity through the Searching of the Setpoint Curve.
SEREA 2017, Seminario Iberoamericano de Redes de Agua y Drenaje	Bogota, Colombia	Optimización de costes de sistemas de bombeo mediante su regulación a través del uso del concepto de curva de consigna.

Besides, the paper presented for WDSA 2016 has been published in *Procedia Engineering Journal:*

- León Celi, C. F., Iglesias-Rey, P. L., & Martínez Solano, F. J. "Energy optimization of supplied flows from multiple pumping stations in water distributions networks". Procedia Engineering, 186(186), 93–100, 2017.

Moreover, five papers aimed at scientific journals have been generated. One of them has been already published, another has already been accepted for publication and the others have been delivered for revision:

a) C. León-Celi, P. Iglesias-Rey, F. Martínez-Solano, and D. Mora-Melia, "A Methodology for the Optimization of Flow Rate Injection to Looped Water Distribution Networks through Multiple Pumping Stations," Water, vol. 8, no. 12, p. 575, 2016.
b) León-Celi, C.F.; Iglesias-Rey, P.L.; Martínez-Solano, F.J. and Savic, D., "Operation of multiple pumped water sources with no storage". Journal of Water Resources Planning and Management, 2018. (Accepted).
c) León-Celi, C.F.; Iglesias-Rey, P.L.; Martínez-Solano, F.J. and Savic, D., "Minimum energy and pumping cost in looped networks with multiple pumping systems and reservoir tanks through the setpoint curve concept", 2017. (Presented for revision).
d) León-Celi, C.F.; Iglesias-Rey, P.L.; Martínez-Solano, F.J., "La curva de consigna como herramienta para la optimización energética y de costes de los sistemas de bombeo en redes de distribución", 2017. (Presented for revision).
e) León-Celi, C.F.; León-Celi, C.F.; Iglesias-Rey, P.L.; Martínez-Solano, F.J., "Understanding of the setpoint curve," 2017. (Presented for revision).

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