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# Hydraulic modelling of control devices in loop equations of water distribution networks

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ABSTRACT

The simulation of hydraulic behaviour of water distribution networks (WDN) needs to develop and implement a mathematical model that is able to consider a wide range of control devices of complex systems. A literature overview is primarily provided for the solution procedures of steady state simulation of nonlinear pipe network hydraulics. Typical elements of pressure regulating valves are conceptually described and differentiated into their functional characteristics to incorporate their hydraulics in the simulation model. They are explored by considering their possible topological positions and operating states. A novel efficient methodology using an *unknown head-loss function* is initially presented for the hydraulic simulation of network flow problems containing static and/or dynamic closed pipes. Closed pipes can be mainly obtained in the distribution networks either by turning off the isolation valves at a pipe segment or as a result of the operating state of unidirectional control devices depending on the pressure distribution in the pipe network. Thereupon, this approach is extended to integrate the control elements of pipe networks such as check valves, pressure reducing (PRV) and safety valves as well as booster/pumping stations. An iterative numerical algorithm is applied to solve the loop equations using the Newton-Raphson method for the linearised energy equation, where Hardy-Cross technique is locally used to correct flow rates of loops containing closed pipes in the iteration procedure. The developed hybrid approach demonstrated robust and very fast converging behaviour for real-world pipe network applications. Moreover, it can consider a variety of combinations of control devices in different network configurations. Several empirical head loss formulas can be additionally used in combination with the commonly known equations such as Hazen-Williams and Colebrook-White head loss formulas. The application of the algorithm will be briefly demonstrated by discussing some simulation results from example and real world large scale WDN.

## 1. Introduction

Pressure/flow regulation in water distribution networks (WDN) is a significant concern for water utilities. Effective pressure/flow control throughout pipe networks is essential to ensure rational sufficient service levels to customers for daily fluctuating demand patterns. Simulation models are applied to estimate the distribution of pipe flow rates and residual nodal heads (pressures) within pipe networks, in which these hydraulic parameters have to be computed for different loading and operating conditions. For implementing pressure/flow regulating devices, it is necessary to make use of comprehensive modelling capability ensuring reliable operation strategies under various demand patterns with existing operational constraints.

Advances in control devices can support the technical improvement of the structural and hydraulic integrity, transparency, and the operational efficiency of the WDN. Hence, a number of pressure/flow control devices can be necessary to be implemented by the real world large scale distribution networks in various configurations (topological positions) and in different combinations to achieve an efficient pressure management throughout the network [103]. Operational management of large scale WDN may impose to control many activities such as switching of pumps/boosters, balancing storage capacities via several different pressure/flow regulating valves. The control elements being equipped with non-return valves are called unidirectional valves that cause to closed pipes in the network as a variant of their operating states. There are several studies for steady state pipe network simula-

*Abbreviations:* BOS, Booster Station; CHV, Check Valve; DFS, Depth-First-Search is a graph theoretical algorithm that gets an interconnected rooted tree from looped networks; ET, Elevated Tank (Storage facility); FCV, Flow Control Valve; FLV, Float Valve (Altitude Valves); HPZ, High Pressure Zone; LPZ, Low Pressure Zone; PBV, Pressure Breaker Valves; PDV, Pressure Dependent Control Valve; PFV, Pressure Relief Valve; PRV, Pressure Reducing Valve; PS, Pumping Station; PSV, Pressure Sustaining Valve; TCV, Throttle Control Valve; TDV, Time Dependent Control Valve; TFV, Tank Inflow Control Valve; UHF, Unknown Head loss Function; VBV, Vacuum Breaking Valve; WDN, Water Distribution Networks; WW, Water Work

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**Nomenclature**

| | |
|---|---|
| $c$ | coefficient of the characteristic curve of a control element |
| $d$ | diameter size of the pipe (L) |
| $g$ | gravitational acceleration (L/T$^2$) |
| $h$ | Head-loss/input of control devices (L) |
| $h_j$ | frictional head-loss of the pipe (L) |
| $h_m$ | minor head-loss (L) |
| $k_e$ | equivalent friction factor (L) |
| $l$ | length of the pipe (L) |
| $l_r$ | number of real (fundamental) loops ($m - n + 1$) |
| $m$ | number of real pipes |
| $m_p$ | number of pseudo-pipes ($n_s-1$) |
| $n$ | number of total nodes |
| $n_s$ | number of source nodes |
| $q$ | flow rate (discharge) of the pipe (L$^3$/T) |
| $v$ | average velocity (L/T) |
| $K_v$ | valve head-loss coefficient |
| Re | Reynolds number |

*Subscripts*

| | |
|---|---|
| $i$ | index of nodes |
| $j$ | index of pipes |
| $k$ | index of iteration steps |
| $s$ | index of source nodes |

*Vectors/Matrices*

| | |
|---|---|
| $\mathbf{h}$ | vector of total head-loss of individual pipes ($m$×1) |
| $\mathbf{q_k}$ | vector of flow distribution in iteration step $k$ ($m$×1) |
| $\mathbf{q_o}$ | vector of initial flow distribution ($m$×1) |
| $\mathbf{u_k}$ | vector of loop flow corrections in iteration step $k$ (($l_r$+$m_p$)×1) |
| $\mathbf{A}$ | $a_{ij}$, incidence matrix ($n$×$m$) |
| $\mathbf{A^T}$ | transpose of the incidence matrix ($m$×$n$) |
| $\mathbf{A_s}$ | incidence matrix of pseudo links ($n_s$×$m_p$), last part of **A** |
| $\mathbf{B}$ | $D_{jj}$, vector of coefficients of linearised head-loss function ($m$×1) |
| $\mathbf{D}$ | $D_{jj}$, vector of coefficients of linearised head-loss function ($m$×1) |
| $\mathbf{H}$ | vector of heads for demand nodes (($n$−$n_s$)×1) |
| $\mathbf{H_s}$ | vector of heads for source nodes ($n_s$×1) |
| $\mathbf{Q}$ | vector of nodal inflows/outflows (supply/demand) ($n$×1) |

tion models coping directly with shutdown of unidirectional valves, since the solution algorithm needs a special handling technique to fulfil the resulting zero flow constraints without altering the network topology or the related loops [16]. Hence, the computer programs have to consider an extended solution algorithm that can efficiently solve the pipe networks including control elements possessing closed operational states in addition to the basic concept for the numerical solution of hydraulic simulation of looped pipe network problems. Besides this complexity, the numerical solution algorithm has to confirm rapid and unique convergence characteristics, even for large distribution systems involving control devices in sophisticated network topologies and different difficult combinations.

Mathematical modelling of pipe networks is next outlined with the reference to the pressure and flow control devices. The literature survey presents some numerical solution methods that are used to solve network flow equilibrium problems. Typical control valves are conceptually expressed and differentiated into their functional characteristics to include their hydraulics in the developed solution algorithm considering their possible topological positions and operating states. The numerical approach is initially described for the simulation of pipe networks involving closed pipes that can stem from operating states of check valves and other control elements or from the non-eliminable closed pipes connecting different pressure zones supplied by separated source nodes. The algorithm is then extended to incorporate several pressure and flow regulating facilities causing to their own operational head loss or head input that is simply added to the frictional head loss of the related pipes. Pressure reducing valves need to be treated in different manner depending on the topological positions and operational states. Finally, the development of simulation model of a real world complex pipe network is shortly explained and the difficulties based on the water balance due to loosely closed isolation valves between pressure zones. In the algorithm, a common head loss formulation is defined to involve the empirically developed equation which is not employed in the commercially available or common share software packages. The developed hydraulic model includes the WDN of the city of Erfurt located in eastern Germany, formerly in GDR as a socialist state of Europe.

## 2. Survey of related works

Analysis of steady state network hydraulics gained more attention in current research of calibration, expansion and/or upgrading of existing WDN [12,52,55,78,99]. Optimisation algorithms of network design and pump scheduling use extensively simulation models to verify the design parameters, i.e. the system pressure and flow distribution. The water quality modelling and risk analysis of distribution systems need efficient simulations of network hydraulics [90,111]. Iterative solution algorithms are primarily carried out for analysing steady state hydraulic behaviour of WDN, where they are developed using different mathematical methodologies to solve the nonlinear system equations governing the pipe hydraulics. Two basic forms are available to exhibit the network hydraulic equations that are implicitly determined such as in terms of the unknown heads at nodes or in terms of the unknown flow rates in pipes. The loop equations are expressed in terms of the pipe flow rates, therefore, they include the nonlinear energy equations besides the linear continuity equations. The formulation of head equations are essentially expressed in nonlinear continuity equations incorporating the pipe flow rates as a function of the nodal heads. Neither formulation of the equations can be solved directly due to the nonlinearity of pipe network hydraulics. Simulation models of WDN were classified in four distinct types defined according to the selected unknowns that were utilised as the system equations [50]. The solution algorithms based on several Newton-Raphson methods were also classified in four groups [82]. The following literature survey is outlined with particular emphasis to the methodologies considering WDN including control valves.

Martin & Peters [72] developed simultaneous correction steps that could be used by Newton-Raphson method for flows or heads along all the loops or demand nodes. Because of their lesser sized matrix with respect to the nodal head equations, the solution approaches of the loop equations are reasonably more attractive for computational applications, although there existed significant improvement to modify the nodal method for inclusion of control devices in various forms [92]. The next hydraulic network models were developed to analyse simple pipe networks including pressure control valves by using the nodal head equations [42,60] and loop flow equations [60,61]. Besides an improved convergence, it is often pointed out that the convergence rate of the techniques depends enormously on the initial estimates of the numerical approach. A hybrid element formulation [75] was proposed to overcome such convergence problems or an alternative technique combining both equations as Newton Loop-Node method [77] was performed. A comparative study pointed out that some of the convergence problems related to the nodal formulation can be effectively overcome by using formulations of flow and loop equations [6]. The application of a Quasi-Newton-Method with BFGS (Broyden-Fletcher-Goldfarb-Shanno) update indicated a competent solution proposal in which the Newton-Raphson technique was exploited for additional iterations [89]. Thus, a numerical algorithm combining both methods was specified to expose better solution approach for the analysis of large scale WDN.

Linear Theory Method uses a suitable transformation to get a set of linear equations from the loop equations [108]. This approach was extended to include pressure reducing valves in pipe networks [66]. The linear theory method consists of a large matrix, and oscillations were sporadically experienced around the exact solution points [74]. Therefore, Nielson [74] developed a practical formulation to assign the loop equations in a compact symmetric matrix by using the linear theory approach for the first iteration followed by iterations of the Newton-Raphson method for flow corrections of loops. The control devices in large scale real world WDN were independently integrated into the Nielson's matrix formulation technique for loop flow equations [8,17,28]. Finite element methods also belong to the iterative solution class of linearization techniques [1,33,106].

The conventional iterative methods for nodal heads ($h$) and loop flows ($q$) were re-examined to improve the $h$-Newton-Raphson solution procedure [97]. They reported that their procedure led to a simplified algorithm and more accurate determination of the Jacobian matrix which accelerated the convergence. A unified framework was employed to derive simultaneous equation algorithms such as Newton-Raphson and Linear Theory, in which this approach was characterised as Newton-Raphson Loop Flows (NR-LF) [102]. In order to compare the computational efficiency, two solution algorithms, the NR-LF and the Newton-Raphson Global (NR-GA) known as global gradient algorithm (GGA) were executed for pipe networks with different sizes and various complex topologies to notice the related computer run-time, where both approaches made use of the linearisation of Newton-Raphson technique for hydraulic equations [37]. In the absence of control valves and pumps, the computational results pointed out that NR-LF showed slight superior performance with respect to NR-GA, however the advantage of NR-LF tends to slow down by increasing the complexity of network topology, but both algorithms led to almost identical performance in case studies of very complex topology. The algorithm efficiency of loop method was improved by avoiding the redefinition of initially selected network loops for incorporating the control devices [7]. They used an efficient procedure determining the loops based on minimum cycle basis of graphs [64] to achieve a highly sparse matrix. Novel efficient null space algorithms were applied to describe the hydraulic analysis of large-scale WDN [4]. Since the linear system of equations within a Newton-Raphson method belongs to the class of sparse saddle point problems, both the structure and sparsity of the equations were exploited to propose more efficient algorithms.

The topological properties of a pipe network were described by using the linear graph theory to analyse the flow and pressure distribution throughout the WDN [56]. The network graph was drawn by connecting the supply (source) nodes to the reference node. An alternative tree selection scheme was recommended [65], where the specified pressure drivers were all put into a tree and the specified flow drivers into a co-tree. This approach was then improved because of its weak convergence characteristics [29]. Unfortunately, the numerical algorithm required a good initial solution vector to speed up the convergence rate for both variants. A different model formulation is provided by selecting the tree of graph by taking a separate reference node that is connected to the all nodes of the distribution network [54]. The connections between the reference node and the nodes of the network were referred to branches that were included in the tree. The connections of the network (pipes) were defined as chords that were not incorporated in the tree of the system graph. These properties make the proposed method different from other linear graph methods. In addition, the tree includes *fictional pipes* to contain the external flows of the pipe network. The flow resistance was expressed in terms of pipe flow, and the degree of freedom of a component was taken to be the flow through it. This extended linear graph theory approach was modified to integrate additional network components such as flow control valves and tanks, and expanded to perform for extended period simulation [18].

Based on the terminology of graph theory, the Jacobian matrix was arranged by partitioning the loop matrix into the tree and co-tree pipes [31]. Pseudo-pipes were additionally prepared by connecting the consumption nodes with the source nodes to determine the flow rates of the tree pipes. The dimension of the solution matrix was decreased to the number of co-tree pipes, in which the matrix was solved via the Newton-Raphson method with a degree of correction ($\gamma \leq 1$) for the iteration steps. By using the network flow equations, the model formulation was remarkably developed by combining concepts of the graph theory and matrix partitioning [85]. The co-tree pipe flow rates identified the independent variables. The size of governing system of formulated equations could be decreased to a smaller set revealed by chord flows. The solution scheme was efficiently established on the reduced storage memory constraint and short execution time. The co-tree method was reformulated to overcome some cumbersome steps in the implementation [45], where it simplified the procedure by manipulating the incidence matrix into trapezoidal form: a lower triangular block at the top representing a spanning tree and rectangular block below it representing the corresponding co-tree. This reordering made the co-tree method competitive with the global gradient algorithm in certain settings.

An approach involving optimisation methods was presented or the solution of the hydraulic equilibrium of the pipe network analysis that can be expressed in terms of a nonlinear convex cost network flow problem [34]. They introduced the terminology of *Content*- and *Co-Content*-model corresponding to the nodal head and loop flow equations respectively. For minimisation of the content model, a hybrid concept was presented for the formulation of a system of equations involving the flow rates and nodal heads [101]. The system formed of partly linear and partly nonlinear equations was solved using the gradient algorithm to find simultaneously the unknown flow rates in the pipes and the heads at the nodes. This global gradient algorithm (GGA) was extended to implement the pressure regulating devices [88]. The GGA were already integrated into the most popular, common share computer package, Epanet2 [87]. Recently, some attempts tried to replace the linear solver with seven different modern multi-core capable solver to test small, medium, and large sized test pipe networks [23]. The aim of the study was to reduce the computation time that could produce promising results in a theoretic research environment, but it failed in practical engineering applications. The GGA method was modified to account for unidirectional devices such as flow control and check valves [51]. The adjustment of the energy balance equations was necessary to involve flow control valves. Check valves were considered as a special case of flow control valves. The unidirectional devices equipped with check valves such as pumps were implicitly treated as check valves in cases of reverse flows. The zero flow problem caused by shutdowns of such devices was solved by iteratively adjusting the minor loss coefficient of the valves outside the GGA, but without modifying the key elements of its equations. A novel modelling technique was presented for integrating unidirectional control devices in extended-period simulations of water distribution systems, where the hydraulic behaviour of the valves was expressed by continuous functions rather than the mixed discrete-continuous formulation [82]. This was defined by the Karush-Kuhn-Tucker equations for an optimisation problem with constraints which were represented by imposing a penalty function added on the head loss of the related pipe if the preset flow rates were violated for the flow regulating devices. Pressure control devices (PRV, PSV, and variable-speed pumps) were implemented into the network simplex code to compare the computational uniqueness of the pipe network hydraulic solutions of different optimisation approaches [19]. They reported that the Todini–Pilati GGA is more efficient than the convex simplex method for content-model by complex water distribution networks. The equilibrium of the pipe network flows were expressed by convex optimisation techniques in which the objective function in primal form included the content-model as the indefinite integral of head loss equations. The dual form of the optimisation problem is then the co-content-model as the flow equation that was considered as the starting point of the computer program code.

An alternative way to cope with the network flow equilibrium is the concepts of non-cooperative games [46], where principles of the game theory were used to solve the equilibrium of general network flow problems. In the case of WDN, the pressure and flow regulating facilities can be characterised as the players and the sets of their operating conditions as their pure strategies. The criterion of the non-cooperative games is automatically satisfied, since operating states of the control elements certainly depend on the pressure and flow distribution in the pipe networks. The control devices actually attain many alternative operating strategies. The basic analytical issues arising in every application of the non-cooperative game theory are the existence, uniqueness, and stability of equilibrium points. In the mathematical literature, the equilibrium problem of dynamical systems involving non-cooperative players is known as *Nash equilibrium* problem, where the feasible sets of the players may depend on the operating strategies of other control devices. Then, the optimisation approach of the content function led to variational equilibrium problems [27,79]. Subsequently, the variational equalities and/or inequalities were defined as an equivalent unconstrained minimisation problem [81]. Chua & Dew [32] expressed how the steady state simulation of a gas transmission network can be converted into a variational inequality optimisation problem subjected to a set of inequality constraints of the maximum outlet pressures and flow rates of the compressors. The correct settings of flow control valves were determined by satisfying the Nash equilibrium for pipe networks [39]. The valve head losses were treated as optimisation variables and were estimated with a gradient-based algorithm minimizing the related convex variational problem. The formulation of the optimisation problem can be alternatively expressed as a mathematical model with equilibrium constraints [80,91]. The necessary and sufficient optimality conditions of such problems were systematically demonstrated, where the proof of the existence, uniqueness, and the sensitivity of different solution algorithms is magnificently exhibited [57]. The existence of the Nash equilibria was presented for non-cooperative network flow problems [68]. Such optimisation problems can be iteratively solved by different versions of Newton method.

Water supply networks are subject to a wide range of physical loads and operational requirements. Hydraulic and structural capacities of pipes and control devices are unfortunately decreased over time. Therefore, pipe break, mechanical failures and/or leaks can occur throughout the network system [105], where they generate very high pressure by transient (water hammer) conditions that are often more than three times of normal operating heads. Similar conditions can also occur by improperly closing/opening the pressure/flow regulating devices of the networks, e.g. rapid valve closure, pump start-up/shutdown, hydrant flushing, action of control valves. Reliable transient analysis is necessary to take countermeasures to the severe unsteady flow conditions in WDN as follows [63]:

1. by rational selection of the required pipe size and strength,
2. by logical sizing the surge-suppression equipments, and
3. by intelligent specification of the system operating rules.

Over the last three decades, a number of modelling techniques were developed to simulate the unsteady state transient conditions in the pipe networks. The method of characteristics (MOC) is the mostly used approach to simulate the transient hydraulic analysis of the pipe networks [48,49,71]. It was also experienced that wave characteristic method (WCM) is able to solve transient pressures and flows in WDN including the effects of pipe friction as well as MOC [43,107]. WCM treats transient conditions by using the pressure wave characteristics. A selective literature review of leak detection methods based on transients was offered with the purpose to describe the state-of-the-art in the area, to provide a degree of historic perspective, and to categorize the major themes among the current and past works [35]. Another review tried to classify the developed leakage management methods into three groups such as leakage assessment and detection methods, and control models, in which the objective was to identify the current state-of-the-art in the field and to make recommendations for future work [84]. A recent work reviewed the previous publications associated with pressure control and energy management for the minimisation of leakage in WDN [2], other recent studies on leakage management via pressure/flow regulating devices are also available [3,25,36,38].

In the modelling terminology of WDN, the conventional techniques supposed that the service pressure level is sufficient to supply the nodal water demand throughout the pipe networks as given in the simulation data. This methodology is known as Demand-Driven Approach (DDA) that is stated in this study. Alternatively, the Pressure-Driven Approach (PDA) was developed to simulate the network hydraulics under pressure-deficient conditions [94]. The definition of PDA can be expressed as inverse problem formulation encompassing variable nodal demand values that result from the estimated nodal heads of the pipe networks. In the last two decades, a number of researchers developed many different algorithms that were applied to solve the pressure-driven pipe network problems [9,44,53,58,59,93,109]. Pressure reducing valves were integrated into the hydraulic simulation of pressure-deficient WDN [95].

## 3. Regulations in pipe network hydraulics

### 3.1. Principles of pressure/flow regulation

The valves are available in many different types and produced by a variety of materials employed in pipe networks. Consequently, each valve type comprises its own specific functional characteristics relying on the network topology. Therefore, the selection of the valves has often special engineering impacts on the pipe networks. Some criteria of control devices have to be particularly considered by the design of pipe networks, e.g. material suitability and availability, extremities of design pressure, velocity and temperature changes, topological and hydraulic constraints for facility operation and maintenance as well as the cost of implementation, operation, and maintenance. Valves are important components of distribution systems, while they regulate either the pressure or the flow in pipe networks. They may be responsible for starting and/or stopping the flows, controlling discharges or pressures, preventing back flows, or relieving the excessive pressure in WDN.

The valves are commonly manufactured in five main structural shapes, such as gate, globe, ball, butterfly, and diaphragm valves [40]. Their functional characteristics are also specified in five categories, e.g. isolation/throttle valves, non-return/check valves, control valves, air and safety valves including float valves, and hydrants 2000 [41]). Control valves are usually implemented to regulate the flows or heads by operating them in partially opened or completely closed states that create head losses or pressure difference between inlet and outlet locations.

*Isolation valves* are installed in WDN to separate a pipe portion for repair, inspection or maintenance. They are commonly modelled either fully/partially opened or fully closed position for a loading condition, however they have to remain in the same position for all steps in extended period simulation (EPS) models that can also involve time-varying operation states of isolation valves. The isolation valves exhibiting time variable operation states (on/off) can be defined in combination with some control devices for individual steps of EPS. In WDN, gate and butterfly valves are the most commonly used valve types for the isolation and/or diversion of flow in a pipe segment, whereas ball valves can also be utilised as isolation valves, however mainly in transmission pipelines with larger diameter sizes. The minor head loss through the valves can be expressed as:

$$h_m = K_v \cdot v^2/(2g) = 8K_v \cdot q^2/(\pi^2 \cdot g \cdot d^4) \tag{1}$$

where $K_v$ = empirical head-loss coefficient of the valve in fully/partially opened form, $v$ = average velocity, $g$ = gravitational acceleration, $d$ = diameter, $q$ = flow rate (discharge) of the pipe. Minor head losses owing to valves are small in comparison with the head loss due to the friction of the flow if the pipe length is large enough. Hence, it is usually neglected in the calculation of the total head losses. Isolation valves can also be used to throttle the flow if the outlet pressure is required to be slightly lower than the inlet head, though the arising transient flows. In large scale pipe networks, the isolation valves between different pressure zones are often not completely turned off, where these valves have to be simulated as *throttle control valves* (TCV). Because of transients arising downstream, they have to be represented in the steady state simulation model with smaller diameter size to get sufficiently accurate results if the pipe segment of such valves is longer than several meters.

Non-return valves are known as *check valves* (CHV) that are usually self-acting and allow the water flow in a predetermined direction, where they are automatically closed to prevent the flow in the opposite direction. There are many types of CHV, such as swing check, tilting-disc check, and wafer check, non-slam (piston type). The check valves are usually used downstream of pumping/booster stations and other pressure/flow control valves to prevent the short-term back-flow [83] and to preserve later the operation of these facilities without interruption. Experimental studies demonstrated that tilting-disc check valves were especially effective by damping the positive pressure of the water hammer [70]. It is then recommended to implement check valves in the outlet pipe of elevated storage facilities.

In WDN, the pressure regulation can occur in three main different ways, where it is adjusted by installing the appropriate control valves in the pipe network such as [13,14]:

1. downstream pressure regulation,
2. upstream pressure regulation, or
3. reduction of pressure difference.

Pressure control devices are typically necessary to regulate downstream the district pressure to a preset certain service level and to meet the water demand of the consumers at sufficient nodal heads. The most known downstream pressure regulating facilities are the *pressure reducing valves* (PRV), the *pump/booster stations* (BOS), and *pressure relief valves* (PFV). Control devices are sometimes installed by a pipe segment connecting two different pressure zones to maintain the nodal heads in the upstream pipe network (high pressure zone, HPZ) in an acceptable range if it is hydraulically possible. Such pressure control facilities are known as *pressure sustaining valves* (PSV). In some cases, pressure regulating facilities are implemented between two different pressure zones to modulate the downstream flow rates in the connecting pipe segment by creating the required head loss or the appropriate pressure difference, when the flow rate exceeds the preset values. Such facilities are called as *flow control valves* (FCV).

If the pressure control facilities are activated in a pipe network, the pressure regulation is achieved through two basic principles of the pipe network hydraulics [14]:

- The pressure is either reduced to a relative constant outlet head of the facility like a PRV or PFV. The downstream head of a PFV is constant if the pressure difference to be dissipated does not exceed the facility capacity, otherwise it is variable and reduced only by the valve capacity that is limited to a certain preset pressure value.
- The pressure regulation is governed by a relationship described by a head-discharge curve, whereas the outlet head is predominantly variable. The best simple example is the minor head loss function for isolation or throttle valves as shown in Eq. (1). Head input is provided by pump/booster stations (BOS) that are embodied by their individual pump characteristic curves. Input of the pump energy is necessary only if the inlet head is lower than the preset values for starting the booster pump. If the facility ought to control the minimal pressure in the upstream pipe network segments, the outlet discharge is to be adjusted such that the possible upstream pressure is to be hydraulically sustained by PSV which requests a pressure measuring device at the reference node when the head at that node is lower than the preset values. The PSV is recognized as the *excess flow regulator* [13,67]. The flow control facility requires the implementation of a discharge measuring device (flow meter) upstream of the connecting pipe segment to reduce the outlet discharge when the flow rate exceeds the pre-specified maximal value in the pipe segment of FCV. It is obvious that the outlet heads of such facilities are mainly variable in any case of the operational state of the valve. Pressure/flow regulation of PSV and FCV has to be governed by a facility characteristic curve describing head-discharge relationship similar to the pump curve.

In WDN, the outlet head of individual control valves actually depends upon the hydraulic operational state of the related valve. Reasonable hydraulic equilibrium can only be realized, when a stable head and flow distribution is truly achieved throughout the pipe network which accordingly reveals the operational states of individual control devices. In this context, the distribution of pressures and flows in the pipe network is directly coupled with the operational states of the pressure and flow regulating facilities, where this relationship can only be iteratively determined.

Air release and vacuum breaking valves are principally installed at the high points and hydraulic high nodes of the supply systems to prevent negative pressure and cavitation in pipe networks [26,86]. *Vacuum breaker valves* (VBV) allow air to enter into the pipe network. Therefore, the downstream flow of the valve will be hydraulically turned to free surface flow condition similar to an open channel flow if VBV is activated. The valves release the air from the pipe network if the normal operation condition is to be maintained. These valves can be simulated in active state for certain loading conditions, whereas it is to be updated for each time step of EPS.

The hydrants are usually implemented as underground and/or stand-post hydrants coarsely in a distance of 100 m to withdraw water for tests or fire fighting in the related supply area. In the steady state network simulation models, they are just relevant for their water consumption to be considered if the simulated loading conditions enclose such situations.

### 3.2. Examples of control devices

The design and implementation plans of flow regulating facilities such as FCV and PSV have to consider the possible maximum and minimum inlet heads as well as the possible maximum and minimum discharges in the pipe segment. The needle/plunger valves are particularly recommended to install by flow regulating facilities (FCV, PSV), since these valves have a very large operational capability in a relationship ranging from 1 to 20 (1:20), e.g. if the maximum discharge is set to 40 l/s in the case of off-peak time to fill the downstream storage facilities, this can be throttled up to 2 l/s by peak demand patterns to sustain the required upstream service pressure. These facilities can be mechanically devised by one of two different acting actuators, either hydraulically self-acting wheel or electrically automated operated actuator, recent developments were summarized by Vicente et al. [103]. The relationship between flow and head loss is normally given by partially closing mechanism of the valve regulation, where it can be determined by experimental measurements and/or simplified mathematical function like a pump characteristic curve [5,69] as follows:

$$h = h_o - c\ .\ q^2 \tag{2}$$

where

$$c = |h_{\min} - h_{\max}| / (q_{\max} - q_{\min})^2$$

The head input of a pump (PS, BOS) or the head loss of PSV can be simply simulated by Eq. (2), whereas the characteristic curve of FCV needs to be the inverse function of this equation.

In WDN, *float valves* (FLV) are often used to close automatically the flow of inlet pipe into storage facilities (elevated tanks or service reservoirs), when the storage capacity reaches its maximum level (tank is full), otherwise the valve stays open, i.e. the valve is inactive. It is obvious that such valves are simulated as open/closed (on/off) for single loading conditions but for extended period simulation (EPS) generally positioned either as *pressure dependent* (PDV) or *time depended* (TDV) valves. EPS necessitates the input of maximal and minimal pressure level at a reference node (level of a storage facility) to close and reopen the valve respectively.

Because of energy savings in pumping systems and water conservation in general, the overflows of storage facilities are undesirable phenomena in operational schemes of the supply systems. Therefore, it is intended to prevent such conditions by implementing one of two possible tank overflow control valves in the inlet pipe:

- Inlet pipe of the tank inflow is closed completely by means of a float valve called PDV, when the storage capacity (level) reaches its preset values at the reference node. In some reference books [22,104], this type of control valves are called as *Altitude Valves*, but in practice *Float Valves* (FLV) are installed to control the overflow of the storage facilities [8,14]. Herein, it is called FLV if it is installed in the inlet pipe of the storage tank; PDV if the pipe to be closed is far from the storage facility, but it is automatically controlled; and TDV if it is closed by dispatcher without relating the valve control to the storage level in the simulation model for specified time intervals. This type of control valves (TDV) can often occur in large scale pipe networks where it is unfortunately not implemented in the available software packages. Examples for these types of control valves are illustrated in Fig. 1 encompassing two different connections for the pump station (PS2).
- Inlet pipe of the tank inflow is controlled by implementing a FCV that reduces the discharge according to the storage level of the tank. In this case, the inflow is not completely closed, but it can be decreased up to the minimal flow of the valve setting such as 1:20 of the possible maximum inflow of the tank. This type of valves is not exclusively characterised in the literature, hence it will be herein defined as *tank inflow control valve* (TFV).

In some cases as shown in Fig. 1b, additional head losses can be caused by inlet pipes, where it can only be simulated by a fictive *Pressure Breaker Valves* (PBV) that may have two (active/inactive) operating states. The excess heads will be dissipated to the preset values by the active valve.

Safety valves are used to prevent the unexpected rising pressure above the preset intensity in a pipeline or by inlet of a storage facility to decrease the inlet pressure. The safety valves are known as *pressure relief valves* (PFV) that are primarily installed in pressure zones without storage facilities and parallel to a pressure reducing (PRV) facility by the diversion pipe segment from main feeders for the case that PRV fails. Further, an appropriate use of PFV can protect the distribution networks from the damages of excessively high or low pressures resulting from transient flows [110]. The valve (PFV) is wide open (active) when the upstream pressure exceeds the predetermined head, otherwise it remains closed (inactive/passive). These valves can be used for bi-directional control of excessive pressures if it is hydraulically possible. In activated case of PFV, the downstream head is constant, when the amount of pressure difference is less than the head loss capacity of the valve, otherwise it is variable and reduced only

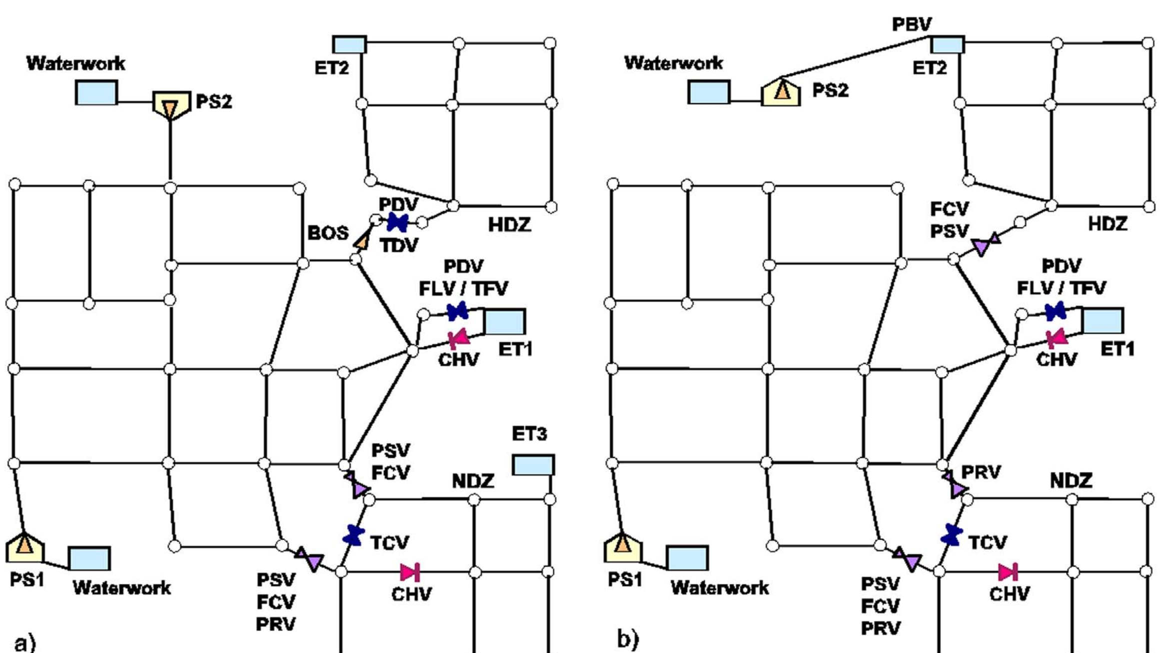


**Fig. 1.** Implementation of possible control valves in different network configurations.

by the maximal valve capacity, since the capacity of these safety valves (PFV) is limited to the maximal values.

By simple WDN, pressure vessels are normally used downstream of pump stations to prevent the high pumping pressure in pipe network during off-peak time, where the simulation of pressure vessels is dependent on the storage capacity and air volume of the vessel, the reader is referred to the literature [83,98] for details.

## 4. Model concept of pipe network hydraulics

### 4.1. Preliminaries

Simulation models of pipe network hydraulics consist of some assumptions that simplify the networks and the representation of the physical phenomena and reduce the computational efforts, e.g.

- small diameter sizes are principally neglected,
- parallel pipe segments are mostly replaced by an equivalent diameter size,
- the nodes connected with short lengths are embodied by a singular node, and
- the water consumption (demand) along the pipes is conventionally considered as it is withdrawn from the adjacent nodes.

Therefore, modelling of WDN regularly includes some precisely known and some imprecise/uncertain parameters. Best examples of uncertain parameters are the diameters and roughness/friction coefficients which are difficult to determine for ageing pipes and their fittings, and the nodal water demands that significantly vary in relation to the number of consumers connected to the distribution network. Although there are a variety of studies using the tedious process based on calibration model applications [12,78,90], these parameters are principally established through values of empirical experiences or mean values of statistical evaluation.

The head loss and flow rates of pipes are determined by means of steady state simulation of WDN that has at least one source node with known fixed head as a reservoir or elevated tank or functional head as a pump characteristic curve. The source nodes are the supply nodes ($Q_i < 0.0$), the other nodes are called demand nodes ($Q_i \geq 0.0$). Since the characteristics of isolation or control valves are defined with their pipe characteristics together, each pipe may incorporate only one control device to be simulated. The existences of a unique numerical solution was mathematically confirmed for hydraulic network equilibrium [21], if every node of the pipe network was defined by a fixed head or by a fixed amount of supply/demand or by a function of head-discharge.

Due to demand fluctuations and pump scheduling, the simulation model of dynamic hydraulic behaviour of WDN requires spatial and temporal discretisations [11]. Therefore, dynamic simulation models are usually carried out in equidistant time steps assuming that the inflows and outflows of the system remain constant during a time step. Hence, the dynamic simulation of WDN is called extended period simulation (EPS). Pump schedules are primarily determined by supposing that the pressure levels of storage facilities change at the end of individual time steps. Small changes in storage levels are neglected to prevent enormous computational attempts if they are within a discrete spatial slice of the storage levels within the optimisation models for pump scheduling.

### 4.2. Topology of Pipe Networks

Pipe networks are mainly represented by graph theory including junctions (nodes) and edges (pipes) which are symbolised by numbers. An edge begins with a node number and ends with another node number. The coordinated form of the nodes and pipes is called *system graph* that is rooted at a source (datum) node, where it can be represented with an incidence matrix describing the connectivity. The matrix of a directed graph with $n$ nodes and $m$ pipes can be exhibited by $\mathbf{A}=a_{ij}$, where $i$=1, 2, ..., $n$ and $j$=1, 2, ..., $m$ as $a_{ij}=+1$ for beginning node and $a_{ij}=-1$ for the end node of the pipe, otherwise $a_{ij}=0$. Hence, the system graph of a pipe network is commonly assessed as a linear digraph that involves definitely an interconnected configuration of a finite number of pipes characterised by a specified length, diameter size, and roughness/friction coefficient.

The graph theoretical definitions of pipe networks were summarized as follows [16]:

- The connectivity is confirmed by determining the existence of a spanning tree that is a sub-graph rooted at a source node and connecting whole nodes without any circuit.
- The Depth-First-Search (DFS) method can be used to obtain a spanning tree from looped pipe networks [100]. The pipes that are involved in the spanning tree are called branches (tree pipes), in which the remaining pipes are assigned to chords (non-tree or co-tree pipes). The spanning tree has exactly ($n-1$) pipes connecting $n$ nodes.
- An individual closed path within the system graph can be generated by inserting any chord into the spanning tree, where the individual closed paths represent the real loops (fundamental circuits) of the system graph.
- Any one of the loops is independent from the other cycles, since every loop includes a unique chord that is not enclosed in any other real loop. Number of real loops are then equal to number of co-tree pipes ($l_r=m-n+1$). Correspondingly, pipe networks with multiple-source nodes ($n_s \geq 2$) include ($m_p=n_s-1$) pseudo-pipes (virtual/fictional edges) linking the datum node with other source nodes, thus the number of pseudo-loops is equal to pseudo-pipes, $m_p$.
- A number of methods can be used to explore the shortest paths [30], minimum spanning trees [24] or minimum cycle bases [73] in the system graph beginning with one end node and ending at another end node of a co-tree pipe.

### 4.3. The proposed model concept

Numerical solution algorithm of the proposed model uses the special matrix formulation concept based on the loop equations developed by Nielson [74]. For the simplicity, pipe related vectors are described herein by small characters such as head loss ($\mathbf{h}=\mathbf{D}\cdot\mathbf{q}$) and flow rates ($\mathbf{q}$) and the nodal related vectors by capital characters such as heads of demand nodes ($\mathbf{H}$) and source nodes ($\mathbf{H_s}$) and nodal supply/demand ($\mathbf{Q}$).

The pipe frictional head loss function is expressed in common formulation that can consider any type of known empirical equations or mix of them or own developed similar equations:

$$h_j = \alpha \cdot l_j \cdot \lambda_j^{\beta} \cdot q_j^{\varepsilon} / d_j^{\delta}, \quad j = 1, \ldots, m \tag{3}$$

Constants of the head loss equation (α, β, δ, ε) have different values depending upon the selected empirical equation employed to compute the frictional head loss of pipes, e.g. Hazen-Williams equation ($\alpha$=10.68, $\beta$=−1.852, $\delta$=4.87, $\varepsilon$=1.852 and $\lambda = C_{HW}$) after Bhave [20] and Darcy-Weisbach equation ($\alpha=8/(\pi^2 g)$, $\beta$=1, $\delta$=5, $\varepsilon$=2) and the friction coefficient λ is to be calculated by Colebrook-White equation, such as

$$\frac{1}{\sqrt{\lambda}} = -2 \cdot \log\left(\frac{2.51}{\mathrm{Re}\sqrt{\lambda}} + \frac{k_e}{3.71 \cdot d}\right) \tag{4}$$

in which $l$=length of pipe, Re=Reynolds number and $k_e$=equivalent friction factor. There are many explicit solution approaches for Eq. (4), in which the implicit solution is iteratively determined. According to the Kirchhoff's circuit law, the algebraic sum of head losses ($h_j$) of the pipes along any loop $L$ is equal to zero, where it is known as energy conservation equation:

$$\sum_{j\in L} h_j = 0, \quad \forall\, L \in (l_r + m_p) \tag{5}$$

The Kirchhoff's nodal rule is recognized as mass conservation or continuity equation, i.e. the algebraic sum of flow rates meeting a node is equal to zero, also at any node sum of inflows equal to outflows, Eq. (6) in matrix form, where nodal supply/demand (Q) is defined as inflow/outflow, respectively. Other relationships were already explained elsewhere [16] for Eqs. (7) and (8):

$$\text{Continuity} \quad \mathbf{Aq} = \mathbf{Q} \tag{6}$$

$$\text{Compatibility} \quad \mathbf{D\,q} + \mathbf{A^T H} = -\mathbf{A_s^T H_s} \tag{7}$$

$$\text{Head loss} \quad \mathbf{h} = \mathbf{D\,q} = \mathrm{f}(\mathbf{q}) \tag{8}$$

where $h_j = H_{i1} - H_{i2} = D_j q_j$ and $D_j = c_j\ |q_j|^{\alpha-1}$

$\mathbf{D} = D_{jj} = \mathrm{diag}[D_1, \ \ldots, D_m]$

In order to maintain the mass conservation Eq. (6), a *spanning tree* is specified by using DFS approach, because it is simple to compute the initial flow distribution ($\mathbf{q_o}$) of a tree-like network by setting zero flow rates for the chords (co-tree pipes) of the pipe network.

The loop matrix ($\mathbf{C}$) has to satisfy the following two constraints:

$$\mathbf{C}(\mathbf{h} + \mathbf{A_s^T H_s}) = \mathbf{0} \tag{9}$$

$$\mathbf{AC^T} = \mathbf{0}, \quad \text{where} \quad \mathbf{C} \neq \mathbf{0} \tag{10}$$

Eq. (9) represents the matrix formulation of energy conservation equations defined by Eq. (5). In graph theory, the matrices ($\mathbf{A}$, $\mathbf{C}$) in Eq. (10) are both orthogonal and complementary subspaces that are linear operators relating edge quantities to nodal and loop quantities [47].

The flow correction vector ($\mathbf{u_{k+1}}$) of the loops is used to update the related flow rates within the iteration process as follows:

$$\mathbf{q_{k+1}} = \mathbf{q_k} - \gamma \cdot \mathbf{Cu_{k+1}} \tag{11}$$

$$\mathbf{J_k u_{k+1}} = \mathbf{C}(\mathbf{h_k} + \mathbf{A_s H_s}) \tag{12}$$

The first iteration (k=0) uses the initial flow vector ($\mathbf{q_o}$) that is determined for the spanning tree. Further variables are denoted as:

$\mathbf{J_k}$=Jacobian matrix for the iteration $k$,

$\boldsymbol{\gamma}$=Degree of correction in iteration steps of Newton-Raphson technique ($\boldsymbol{\gamma}$=1.0 for the first iteration step, $\boldsymbol{\gamma}$=0.5 in the next **i**teration steps).

In addition, the flow rates in the loops including closed pipes (zero flows) are adjusted by using a Hardy-Cross local correction approach allowing a small reverse flow rate varying within lower and upper bounds such as $\varepsilon_{min}$ and $\varepsilon_{max}$ respectively during the iteration process. The head loss of such pipes is estimated by using an *unknown head loss function* (UHF) to avoid the matrix singularity [16].

## 5. Numerical solution algorithm

### 5.1. Basic model with closed pipes

Due to the symmetric matrix of Eq. (12), it is sufficient to save the upper triangular matrix in an array **B** for the solution procedure. The compact form of **B** can be completely represented by storing only the non-zero elements with their indexes and graph theoretical information. Subsequently, the Jacobian matrix is established as follows [15,17]:

- Diagonal elements:

  $b_{ii}=\Sigma\, D_j$, for the pipes $j$ that are elements of the loop $i$.

- Non-diagonal elements:

  $b_{ih}=(-1)^z\, \Sigma\, D_j$, for every pipe $j$ that is an element of loops $i$ and $h$ with a positive (z=2) or negative (z=1) algebraic sign that is obtained by multiplying the signs of the pipe shared by both loops.

In large scale WDN, pressure zones are principally interconnected by some pipes that are closed by isolation valves turned off. The elimination of such closed pipes may cause to some separated network topologies which lead to prepare accordingly separated input network data to solve the hydraulics of individual data set of the unconnected sub-graphs, since Jacobian matrix can only be solved for interconnected system of graphs. In order to permit the users to prepare a common data set for pipe networks interconnected with such closed pipes, a sorting algorithm based on graph traversal technique was developed to label these static closed pipes among pressure zones as *eliminable* or *non-eliminable* pipes by assuring the graph connectivity [16,17]. The labelling technique was achieved by using the degree of connectivity of the related nodes from incidence matrix and the position of the related pipes by the loop matrix. The implementation of this approach in program code made the proposed solution algorithm superior in comparison with Epanet2 for such complex networks. There exist also some attempts to improve Epanet2 by this pitfall, in which a search algorithm was prepared to detect the unintended isolation of nodes by deleting such closed pipes [62].

The flows in *non-eliminable* closed pipes are bidirectional depending on the heads at the adjacent nodes. Control devices (BOS, FCV, PRV, PSV) are mainly equipped with non-return valves to prevent reverse flows, so that they react as check valves which permit the flow only in one direction, i.e. unidirectional flow. Both types of closed pipes are subject to the boundary constraints of zero flows that were solved by allowing negligible values for flow rates to vary between upper ($\varepsilon_{max}$) and lower ($\varepsilon_{min}$) limits [16]. It was experienced that the local flow correction by Hardy-Cross approach in the loops with closed pipes can stabilise and enhance the main iteration process. In each iteration step, head losses of the pipes are principally calculated by considering the iteratively corrected flow rates, while the UHF was developed to estimate the head losses across the closed pipes. UHF uses the advantage of reachability of the adjacent both nodes of closed pipes at least by two paths to compute the related head loss by taking the head difference between both nodes that are also corrected in each iteration step.

### 5.2. Extended model for control devices

In the first iteration step, the head loss of the individual pipes can be simply calculated depending on the provided initial flow distribution, but the activation of the control devices is difficult to be examined due to the lack of nodal head estimates. Therefore, the elevation difference of adjoining nodes with a constant ($c = \pm 2$) was defined as additional head loss/input for the pipes including the control devices [17]. The amount of head loss was determined by a positive sign, then the head input of booster stations has the opposite sign of the head loss, i.e. minus signed values. Hence, it was assumed that the control devices were active at the beginning of the iteration process, if the pipe is not closed due to the estimated initial flow distribution of the pipe network. The closed pipes for the control facilities (BOS, FCV, PRV, PSV, TFV) were considered to have the minimal flow rate ($\varepsilon_{min}$) and head loss ($\varepsilon_o$) in the first iteration step, thereafter they were treated as described in the aforementioned procedure of the basic model.

Subsequently, the additional head loss/input is added to the head loss due to friction and minor losses of the individual pipes if the control device has the active operating state due to the results of preceding iteration step. This process was carried out at the beginning of the next iteration step until the stopping criteria were fulfilled to exit the iteration process. However, there are some exceptions for PRV and additional rules for the flow regulating valves (FCV, PSV, TFV) which need their active operating state to be presented by the additional head loss of the facility. This information is to be stored in an additional array for each flow regulating valve, since the solution algorithm of the loop equations need only the flow rates to be stored, where the heads of the demand nodes are obtained by calculating the head loss of the individual pipes with the flow rates resulted from the actual iteration step, since the heads of source nodes are known as constant or as the value of a pump curve with the actual iterated outflow. If the valve of the flow regulation is active, it means that the proper flow rate has to be estimated by adding the amount of discharge reduced due to the head loss caused by the facility. The head loss is then to be updated for each iteration step to be able to match the amount of flow rate along the characteristic curve. Numerical oscillation problems by FCV can be avoided if the amount of the iterative flow rate is constrained to be within a certain interval of maximal allowed flow rate ( ± 3%), e.g. $q_j \le q_{max} \pm 0.03 \cdot q_{max}$, if the facility is active.

In the model formulation of the nodal heads, the operating states of the PRV can be determined simply by taking into consideration the difference of the estimated heads of adjoining nodes of the related pipe. Unfortunately, the head to be reduced by the valve (PRV) cannot be directly estimated by the model formulation in loop equations, they are then to be treated in a sophisticated way which need to build up and update the Jacobian matrix according to the different topological positions in the pipe network, as shown in Fig. 2. The pipe of PRV can be in a real loop (Fig. 2a), in a pseudo-loop (Fig. 2b) or in a single pipe connecting two pressure zones with one storage facility (Fig. 2c). The head loss of PRV in a pseudo-loop will be algebraically added to the loop equation in the Jacobian matrix for each iteration step if the head at the upstream node is greater than the preset fixed head at downstream node of the facility. A pseudo-loop may include many PRV facilities that can be estimated in similar manner. In the case of single pipe as shown in Fig. 2c, the head loss of the valve will be added to the minor and frictional head loss of the related pipe, similar to the other control valves, if the facility is active. The PRVs in real loops are to be treated in a completely different approach, such that the Jacobian matrix is rebuilt by including the pseudo-loops created for each PRV in real loops such as:

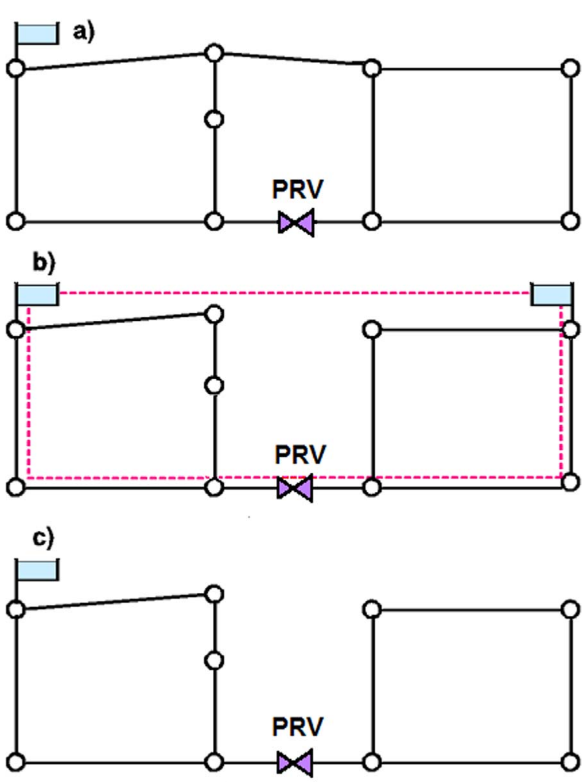


**Fig. 2.** Possible configurations of a pressure reducing valve (PRV) in different topological positions of the pipe network.

- Firstly, a pseudo-loop is built up for the facility (PRV) in each real loop that can involve only one PRV being directly connected to the datum node. If a real loop incorporate two or more PRV facilities that are not shared with any other loop, one of them will be labelled as *attached pipe* and the other facilities as *unattached pipes*. The pseudo-loop will be constructed by connecting the attached pipe (PRV) directly with datum node, the remaining PRV facilities will be automatically enclosed in this pseudo-loop. Fig. 3 illustrates an example pipe network including two real and two pseudo-loops. The facilities shared by two real loops as shown in Fig. 4 are stored in additional two vectors, one of them is used to save the pipes of real loops, the *transformed loops* arising after eliminating the pipes of the control valves (PRV) are saved in the second vector.
- In the second stage, the triangular matrix **B** is rebuilt by replacing the real loops of the individual facility with their related pseudo-loops, the transformed loops are used for one of the real loops as shown in Figs. 3 and 4. The configuration of the transformed loops is illustrated in Fig. 4b. Then, the control valves are started with active operating state, the remaining valve operating states will be verified after the convergence of the numerical iteration procedure. The amount of head loss of the facility is added to the frictional head loss of the related pipe which is equivalent to the head difference between the datum node and preset head of the facility represented by the connecting pseudo-pipe.
- Finally, the flow correction is carried out by the real loops of the control valve (PRV), since the pseudo-loop of a PRV possesses an *imaginary* fixed potential node with a head level corresponding to the preset values of PRV outlet head, where neither inflow nor outflow is permitted into/from this fictional potential node (source node without storage capacity).

The proposed approach to simulate the hydraulics of control devices is outlined in Fig. 5 that show the substitution of loops for PRV in real loops due to the operating states and the integration of UHF for the shutdown of devices in the steady state model of WDN.

By the convergence of the iteration process, each control element can get one of the three operating possibilities as follows:

- Reverse flow in the pipe involving a unidirectional control element,

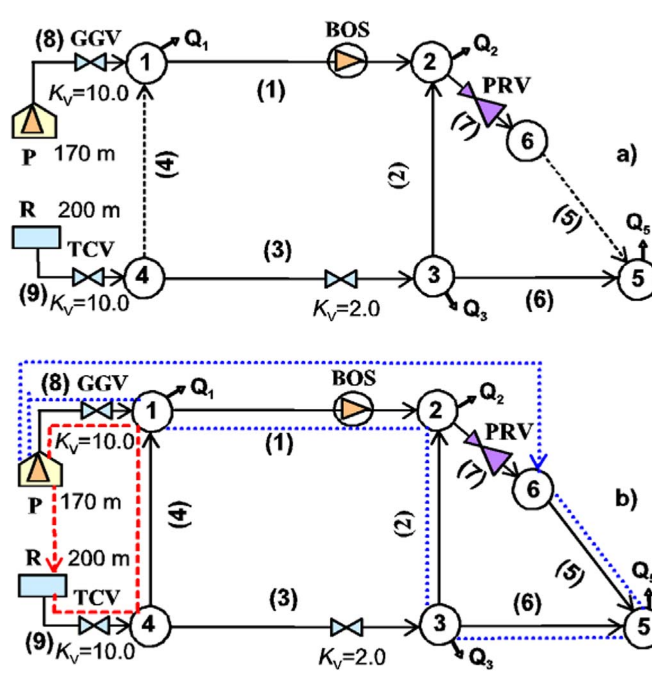


**Fig. 3.** An example pipe network with two real loops and the formation of two pseudo loops.

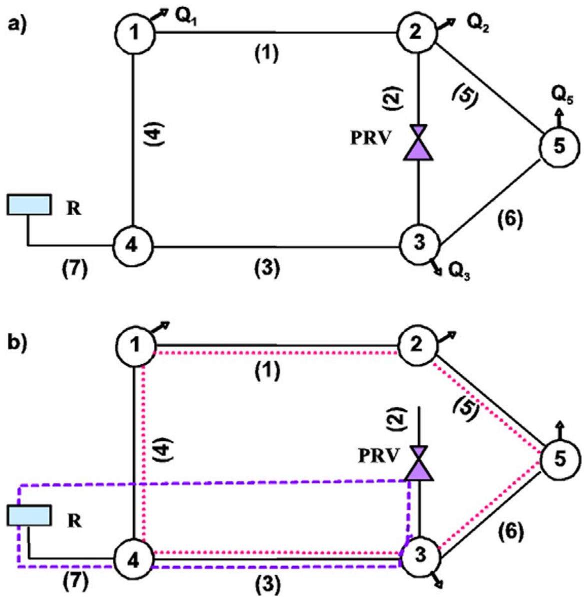


**Fig. 4.** An example pipe network with two real loops and its decomposition into a pseudo and a transposed loop.

which means that the downstream pressure is greater than the pressure at the upstream node. Hence, the pressure/flow regulating facility is closed by means of the downstream installed check valve.

- There exist a lower head/flow condition than the preset values at the reference nodes/pipes. It means that control elements are inactive (passive), because there is no violation for the preset nodal head at the reference node or the flow rate through the relevant pipe for pressure regulating facilities (FLV, PDV, PFV, PRV) or flow regulating facilities (FCV, TFV) respectively. The inactive booster pumps (BOS) and PSV necessitate higher nodal heads at the upstream nodes.
- Pressure reducing or boosting at the downstream node by active operating states of the control devices. In this case, the pressure reduction/boosting or flow regulation is performed by activating the pressure/flow regulating facilities. The downstream nodal head is reduced by PRV to the preset values at the downstream node. If the control element is a BOS, the downstream nodal head is increased by BOS, where the head loss caused by other pressure regulating facilities is added to the frictional head loss of the related pipe.

The developed algorithm can consider the impacts of an upstream control device on the downstream control devices by providing the input data of the control elements successively in the flow direction. Convergence of the iteration process is reached, when the algebraic sum of heads in the loops do not exceed the user provided upper

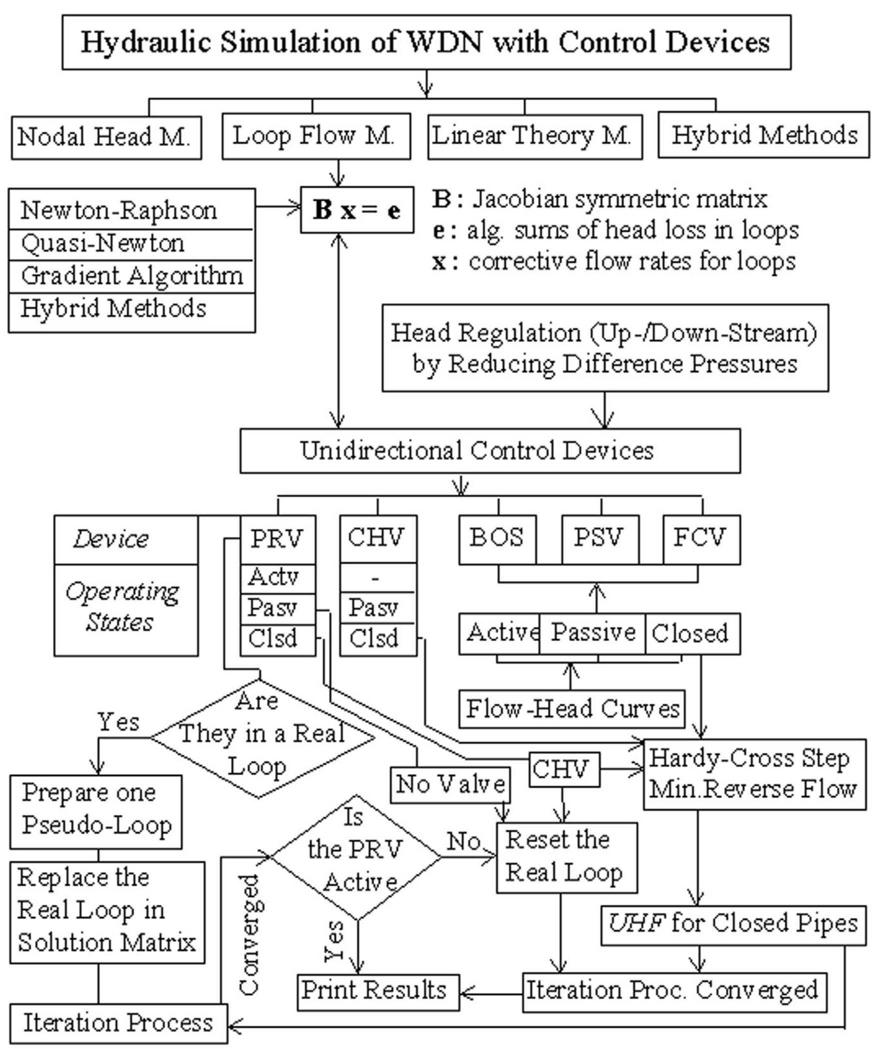


**Fig. 5.** Outline of the proposed algorithm for WDN with control devices.

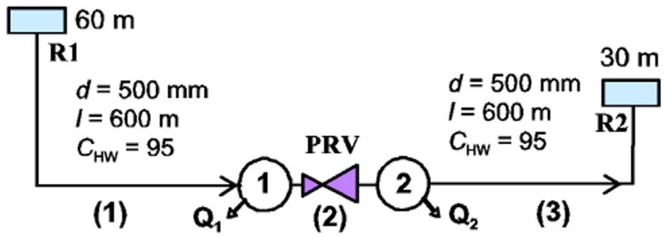


**Fig. 6.** A simple pipe network with a unidirectional control device.

bounds. In the case of the augmented model, the convergence has to satisfy the additional constraints for the reverse flow rate in closed pipes ( < $\varepsilon_{max}$).

## 6. Application to simple pipe networks

One of the simplest pipe networks consists of two source nodes connected with a pipeline that may have a control device, such as illustrated in Fig. 6. The PRV is along the unique pseudo-loop, hence the resulting head loss will be added directly to the head loss equation of the solution matrix. The pseudo-loop consists of the real pipes and pseudo-pipe from R1 to R2 such as in compact form $l_1(-1, -2, -3, 4)$. There exist three operating states of the PRV placed between nodes 1 and 2 enclosing a pipe segment of 2 m length with 500 mm diameter. Due to elevation difference, it is obvious that the water will be reasonably supplied in a particular flow direction from reservoir R1 to reservoir R2. Possible operating states (active, inactive/passive, shutdown) of the valve (PRV) can result from the settings for downstream heads of the valve. Simply, the device is active if the downstream head of the valve was set between both reservoir elevations (60 m > $H_{PRV}$ > 30 m), when nodal demand is assumed to be zero ($Q_1$=$Q_2$=0). The device is fully open (inactive) if the valve setting is over the elevation of R1 ($H_{PRV}$ > 60 m). The valve will shutdown if downstream head is adjusted to be below the elevation of R2 ($H_{PRV}$ < 30 m), also the pipe is closed so that it cause to zero flow boundary constraint which need a special solution approach.

Several computer runs were executed to examine the efficiency and robustness of the developed solution algorithm for the implementation of control devices. For zero demand at nodes 1 and 2, the valve was set to 35 and 40 m for active state which yield to flow rates 322.4 and 468.4 l/s flow rates respectively. PRV coefficient was set to $K_\upsilon$=0.0 for all computer runs. The convergence of algorithm was accomplished for flow rate 582.53 l/s with different valve settings 45, 50, and 60 m for downstream head, where the valve was inactive. The iteration process was interrupted in a few steps by the valve setting of 20 m, since the corrective flow rate in the pipe network tends to zero, i.e. matrix singularity was reached. In order to demonstrate the developed approach, the closed pipe situation was solved for valve setting 20 and the water demand at node 1 (250 l/s) and 2 (50 l/s) with the elevation of 60 m for both reservoirs, the reverse flow rate (0.013 l/s) through the valve was estimated due to the closed valve of PRV with 2.97 m head loss across the pipe as the difference of heads 56.87 m and 59.84 m at nodes 1 and 2 respectively.

An example pipe network with a PRV in a real loop was illustrated to show the efficiency of the solution approach for closed pipes [16]. The real and pseudo loops are shown in Fig. 3 and their matrix representations in Table 1 for two different datum nodes (Pump station/Reservoir) that make possible to carry out some comparisons between the results for various valve settings. First three loops include the same pipes in comparison with the former study [16], but some of them have different directions in the system graph, since the pipes 5 and 7 are presented in Fig. 3 with opposite direction. In addition, Table 1 incorporates two parts that are developed to demonstrate the differences within loops if datum (source) node is changed to root the pseudo-loops in multi-source pipe networks. Main differences particularly occur in the pseudo-loop that is created for the PRV in the real loop of a pipe network including two source nodes (see Fig. 3 and Table 1). The real loop 2 was replaced by the pseudo-loop built for PRV, the pseudo-pipe was directly connected with downstream node 6 of the pipe 7 which is the beginning node of the pipe 5. This pseudo-loop do not include the pipe 7 (PRV), hence the flow rates have to be corrected in the real loop of the valve PRV. Special handling of PRV in loop equations is unavoidable, since it has a constant head at the outlet by an active operating state, where other control devices are simulated with their additional head loss/input in the model.

**Table 1**
Loop data of the pipe network in Fig. 3.

| Datum Node: 10 (P) | Matrix representation for Eq. (12) | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Chords: 4, 5,10 Pipes in Loops (Compact Storage) | Pipes | Real | | | | | | | | | Pseudo | |
| $l_1(1, -2, -3, 4)$, $l_2(-6, 2, 7, 5)$, $l_3(9, 4, -8, 10)$ | Loop | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 |
| | 1 | 1 | −1 | −1 | 1 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| Pseudo-loop with PRV | 2 | −1 | 1 | 0 | 0 | 1 | −1 | 0 | −1 | 0 | 0 | 1 |
| $l_4(5, -6, 2, -1, -8, 11)$ | 3 | 0 | 0 | 0 | 1 | 0 | 0 | 0 | −1 | 1 | 1 | 0 |

| Datum Node: 20 (R) | Matrix representation for Eq.(12) | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Chords: 4, 5,10 Pipes in Loops (Compact Storage) | Pipes | Real | | | | | | | | | Pseudo | |
| $l_1(1, -2, -3, 4)$, $l_2(-6, 2, 7, 5)$, $l_3(8, -4, -9, 10)$ | Loop | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 |
| | 1 | 1 | −1 | −1 | 1 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| Pseudo-loop with PRV | 2 | 0 | 0 | −1 | 0 | 0 | −1 | 0 | 1 | −1 | 0 | 1 |
| $l_4(8, -6, -3, -9, 11)$ | 3 | 0 | 0 | 0 | −1 | 0 | 0 | 0 | 1 | −1 | 1 | 0 |

Computer runs were executed by taking $D_j$=$\varepsilon_o$=1.0E−06 for the critical range of zero flow rates ( ± 1.0E−06 $m^3$/s) that may take place in the initial flow distribution. Zero flows of closed pipes were set to vary within threshold values $\varepsilon_{min}$=0.005 and $\varepsilon_{max}$=0.025 *l/s*. The magnitude of algebraic sum of heads along each loop was considered to be less than 0.001 m ($\varepsilon_h$ < 0.001) for the convergence criterion. Three operating states of PRV were estimated by changing the settings at the valve outlet head such as $H_{PRV}$=149.00 m for active, 159.00 m for inactive (passive), and 135.00 m for valve shutdown (closed pipe) situations. The results of individual runs are presented in Table 2. Two computer runs were executed by varying the datum node as shown in Table 1 for each operating state of the valve. The results in Table 2 verified that the developed approach efficiently works and converged exactly to the same records for flow rates and heads, although the matrix of loops were different in the solution algorithm for active case ($H_{PRV}$=149.00 m). The slightly different nodal heads ($H_i$≤0.001 m) can result from rounding errors within the iteration process. Unfortunately, the convergence rate showed some differences, e.g. 12 iteration steps for datum node with the pump (P, No. 10) and 18 steps for datum node with reservoir (R, No. 20). The iteration process showed similar convergence rate for the loop data with different datum nodes. The results of computer runs for valve shutdown (closed pipe) are not given in Table 2, but it has the results of the reconstructed loop data for the valve PRV changed with its real loop data after the convergence of the algorithm with reverse flow rate. In order to demonstrate the accurate values of flows and heads, the Table 2 containing two runs for the pipe

**Table 2**
Simulation results for the data of pipe network in Fig. 3.

| Data of Pipes | | | Data of Flows | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | $H_{PRV}$=149 m | | $H_{PRV}$=159 m | | CHV | | No Valve | |
| Pipe No. | Length (m) | Diam. (mm) | P $q_j$ (l/s) | R $q_j$ (l/s) | P $q_j$ (l/s) | R $q_j$ (l/s) | P $q_j$ (l/s) | R $q_j$ (l/s) | P $q_j$ (l/s) | R $q_j$ (l/s) |
| 1 | 500.0 | 200 | 102.449 | 102.447 | 103.339 | 103.337 | 102.051 | 102.051 | 103.161 | 103.161 |
| 2 | 300.0 | 200 | −14.714 | −14.714 | 17.638 | 17.638 | −22.041 | −22.041 | 13.978 | 13.978 |
| 3 | 500.0 | 200 | 107.551 | 107.553 | 106.661 | 106.663 | 107.949 | 107.949 | 106.839 | 106.839 |
| 4 | 300.0 | 200 | 74.286 | 74.286 | 74.957 | 74.957 | 73.986 | 73.986 | 74.824 | 74.824 |
| 5 | 400.0 | 200 | 7.734 | 7.733 | 40.977 | 40.976 | 0.010 | 0.010 | 37.139 | 37.139 |
| 6 | 500.0 | 200 | 72.266 | 72.267 | 39.023 | 39.024 | 79.990 | 79.990 | 42.861 | 42.861 |
| 7 | 200.0 | 200 | 7.734 | 7.733 | 40.977 | 40.976 | −0.010 | −0.010 | 37.139 | 37.139 |
| 8 | 300.0 | 250 | 58.163 | 58.162 | 58.381 | 58.381 | 58.065 | 58.065 | 58.337 | 58.337 |
| 9 | 300.0 | 250 | 181.837 | 181.838 | 181.819 | 181.819 | 181.935 | 181.935 | 181.663 | 181.663 |

| Data of Nodes | | | Data of Nodal Heads ($\varepsilon_h$=0.001 m) | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | $H_{PRV}$=149 m | | $H_{PRV}$=159 m | | CHV | | No Valve | |
| Node No. | Elev. (m) | $Q_i$ (l/s) | P $H_i$ (m) | R $H_i$ (m) | P $H_i$ (m) | R $H_i$ (m) | P $H_i$ (m) | R $H_i$ (m) | P $H_i$ (m) | R $H_i$ (m) |
| 1 | 120.0 | 30.0 | 175.184 | 175.184 | 175.118 | 175.118 | 175.213 | 175.213 | 175.131 | 175.131 |
| 2 | 130.0 | 80.0 | 155.664 | 155.665 | 155.278 | 155.279 | 155.836 | 155.836 | 155.355 | 155.355 |
| 3 | 100.0 | 50.0 | 155.333 | 155.334 | 155.737 | 155.738 | 155.152 | 155.152 | 155.657 | 155.657 |
| 4 | 130.0 | 00.0 | 181.595 | 181.594 | 181.637 | 181.637 | 181.576 | 181.576 | 181.629 | 181.629 |
| 5 | 100.0 | 80.0 | 145.185 | 145.185 | 152.497 | 152.497 | 142.883 | 142.883 | 151.806 | 151.806 |
| 6 | 100.0 | 00.0 | 149.000 | 149.000 | 153.860 | 153.861 | 142.883 | 142.883 | 154.172 | 154.172 |
| 10 | 170.0 | P | 177.262 | 177.262 | 177.211 | 177.211 | 177.285 | 177.285 | 177.221 | 177.221 |
| 20 | 200.0 | R | 200.000 | 200.000 | 200.000 | 200.000 | 200.000 | 200.000 | 200.000 | 200.000 |
| | $h_{PRV}$* | | 6.595 | 6.595 | 0.00 | 0.00 | – | – | – | – |
| | $h_{BOS}$* | | −3.676 | −3.676 | −3.667 | −3.667 | −3.679 | −3.679 | −3.668 | −3.668 |
| | Iteration Step | | 12 | 18 | 12 | 18 | 6 | 6 | 6 | 6 |

P: Pump, R: Reservoir.
* Head loss/input.

network without any valve in the pipe 7 that corresponds to the runs after changing the loop of the inactive valve PRV with the related real loop without valve at the related pipe. The change of pseudo-loops with inactive and shutdown operating states of PRV is integrated in the solution algorithm, the user has to control the last results of program code that was developed to analyse the pipe network of the city of Erfurt [17].

The solution matrix **Bx=e** can be constructed equivalent to the matrix of loops, in which **x** is the searched solution vector (corrective flows) for the individual loops, **B** is the array stored as described above in Section 5.1 and **e** is the algebraic sum of heads along each loop. Construction of the solution matrix can be expressed by looking for the common pipes among the loops of datum node with pump station for the non-diagonal elements such as $b_{12}$=(−1, −2), $b_{13}$=(+4), $b_{23}$=(+8), where the sum of head loss derivatives ($D_{jj}$) for individual loops, $l_1 \rightarrow b_{11}$, $l_2 \rightarrow b_{22}$, $l_3 \rightarrow b_{33}$. Thereafter, the values of the first iteration were calculated as $b_{11}$=462.27820, $b_{12}$=−128.09000, $b_{22}$=318.60150, $b_{13}$=1.0E-06, $b_{23}$=36.74436, $b_{33}$=370.34400 and $e_{11}$=−67.27330, $e_{22}$=20.47037, $e_{33}$=−7.323403 with head input by booster $h_{BOS}$=−6.050003 in pipe 1 and the difference of elevations (30.0 m) plus a constant ($c_1$=10.0 m) $h_{PRV}$=40.0 m for initial value for head loss due to PRV in the loop 2. The vector of corrective flows ($\mathbf{u_k}$) will be attained by using a factorization technique for the solution of a symmetric matrix to get the inverse of **B**, such as $\mathbf{x}=\mathbf{B}^{-1}\mathbf{e}$. The Fortran subroutines SSPFA and SSPSL of LINPACK is used by the proposed approach with necessary matrix manipulation routines to provide the solution vertical vector ($\mathbf{u_k}=\mathbf{x}$) for each iteration step. The program code of SSPSL employed Bunch-Kaufman method to offer stable factorizations for the direct solution of symmetric linear equations. The Bunch-Kaufman factorization is a commonly recognized algorithm, this method is already compared with alternative factorization techniques [10]. Further details of the proposed solution algorithm were presented to explain the basic model for closed pipes [16].

In order to verify the efficiency and robustness of the proposed solution algorithm, two additional strategies were integrated in the program code to differently initiate the zero values for flow rates of the pipe with PRV, e.g., 4.0*$\varepsilon_{min}$ and an average flow rate that was estimated by determining the magnitude of maximal pipe flow rate divided by number of pipes of the related loop. It was recorded that the iteration steps and the data of resulted flows and heads were identical with the former described technique. Unfortunately, the stabilisation of solution vector was mainly achieved for the active case of PRV after five iteration steps, since it was observed that reverse flows were resulted in the valve pipe during the first 3–4 iteration steps. This is the pitfall of the developed approach for PRV in the real loops. The advantage of the closed pipe for unidirectional control devices could not be exploited, since the pipe of the valve was not directly involved in the loops of the solution matrix.

## 7. Application to a large scale water distribution network

Real world large scale water supply networks exhibit some different problems in comparison with small pipe networks that are mostly presented as example networks in literature studies. It may take long time to verify the individual pressure zones due to enormous number of isolation valves being defected or often not completely turned off. Accordingly, the mass balance among high and lower pressure zones cannot be adequately estimated because of prevailing transient flow

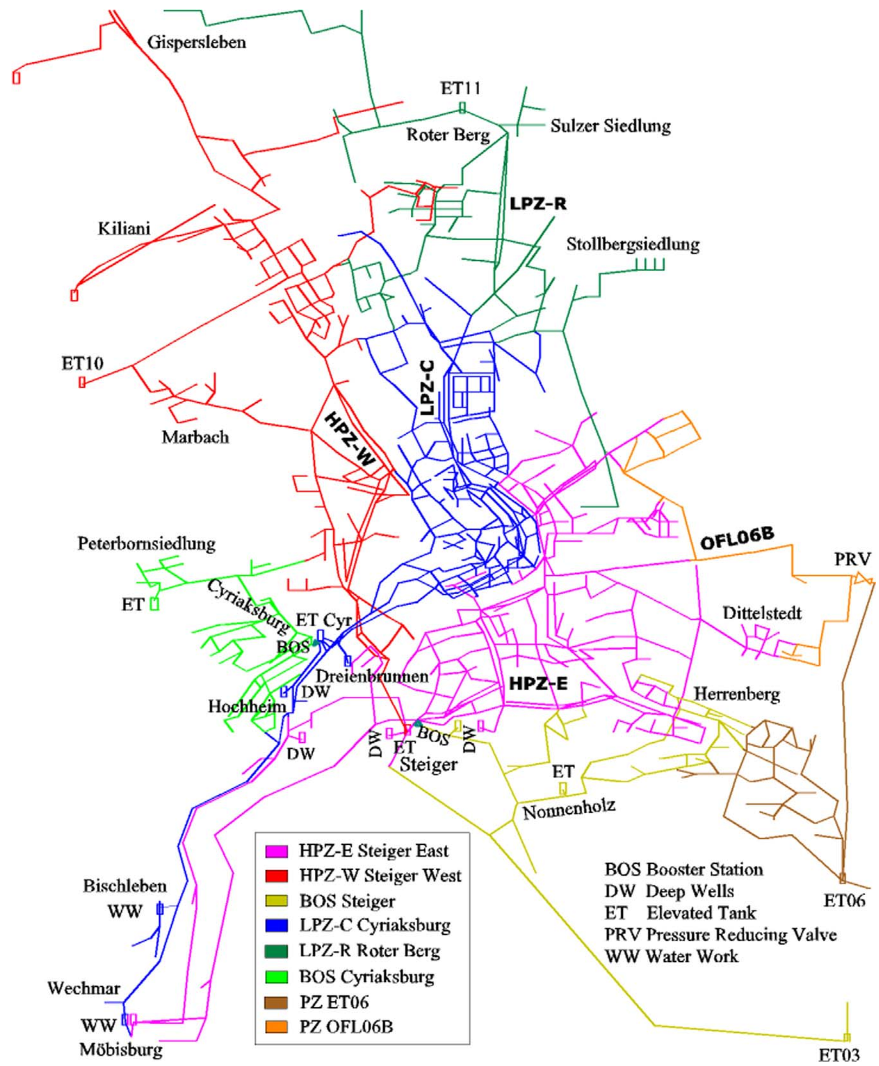


**Fig. 7.** Pressure zones of the water distribution network of the city of Erfurt in 1998.

conditions through partly opened valves which are to be simulated in steady state flow conditions. They are to be simulated principally as a throttle valve (TCV) frequently with smaller diameter size and huge valve coefficient. In some extent, the amount of water losses throughout the pipe network can make it difficult to equalise the water supplied from several water works to the consumer demand of interconnected several pressure zones. They can be unaccounted as evident leaks and/or as droplets of water taps in household connections and in the old pipe fittings. This situation requires a diagnostic approach to identify the causes and their impacts on the estimation of the nodal demands that are essential to develop a coarsely calibrated hydraulic simulation model for the distribution network. Practical experiences showed that the development of the hydraulic model requests some extensions of the commercially available software packages that may cause highly more expenses to the water authorities.

In the beginning 1990 s, it was decided to develop a robust and efficient numerical algorithm considering the special difficulties of a real world WDN that exhibited sophisticated interconnections and very old pipes of different materials manufactured in various eastern European countries over five decades. Some laboratory experiments demonstrated that the standard head loss formulas cannot estimate the real head loss observed in the field measurement. A former version (KANET) of the proposed algorithm (WASNET) was developed at the University of Karlsruhe to analyse the WDN of the city of Erfurt as shown in Fig. 7 [17]. For the old pipes in the city centre, an own head loss formulation was to be included in the simulation model of the pipe network of Erfurt. Therefore, a common formulation like Eq. (3) for head loss equations was expressed to make use of user specified equations in that software package.

The WDN demonstrated in Fig. 7 supplied about $12.0\times10^6$ $m^3$/year billed water to the inhabitants of about 240,000 in the city of Erfurt and its suburban area. The city is the capital of the federal state Thuringia located in eastern Germany. The pipe network contains mainly four pressure zones (two high and two low pressure zones), in which they are also divided into some sub-zones regulated by control elements, such as booster station (BOS) and pressure reducing valve (PRV). The high pressure zones (HPZ) are ET Steiger separated into eastern/western parts and BOS Steiger (Nonnenholz) and ET06 including two parts, Herrenberg and OFL06B that was regulated by PRV. The lower pressure zones (LPZ) are ET11 (Roter Berg) and ET Cyriaksburg feeding directly the city centre and the suburban town Wechmar, the districts Cyriaksburg and Peterbornsiedlung were supplied from ET Cyriaksburg by a BOS. In order to determine the pressure zones, first simulation results with roughly estimated nodal demands were provided to the technical staff of the operational management for controlling the closed isolation valves. Based on the field check-ups, some border displacements among the pressure zones could be performed by the input data. It was possible to carry out simple pressure measurements at the critical nodes to compare with the roughly simulated results, since the head difference between HPZ and LPZ was quite high on some particular nodes, about 30–40 m. After the corrections of the border displacements, the WDN of the city of Erfurt included 1061 nodes and 1266 pipes with nine pumping stations, two booster stations, nine elevated tanks, and 74 isolation valves separating the pressure zones, where the half of them were not completely turned off, since pressure problems limiting the service level were observed in some particular parts of the LPZs.

The reservoirs ET03, ET06, ET10, and ET11 were filled with surface water from the trunk mains of the regional water supply utility, ET03 was used for emergency water supply of the HPZ Steiger and ET06 that is not connected to ET03 in the simulation model. The groundwater was pumped into the pipe network from the water works (WW) and deep wells (DW) around the city centre which were the own utilities of the city authority. The annual demand was roughly met by 1/3 from softened regional surface water and 2/3 from groundwater which was very hard, i.e. it had a high concentration of dissolved salts. This led to supply water with poor quality, since mix of the hard groundwater and soft surface water is chemically more aggressive that mostly causes to the corrosion of pipe materials in the network. The objection of the consumers reasonably increased because of the supply of coloured drinking water leading to deposits and encrustations that were existed in a major part of the old pipe network in the city centre.

The scope of the unique project in Germany was to offer support for the identification of deficiencies and the development of rehabilitation strategies by using system analytical methods applying mathematical modelling techniques for the pipe network which were begun by the water authority after the unification of the two German states in 1990. The methods comprising the hydraulic simulation and optimisation models should be applied to minimise the existing high water losses (about 45%) and to secure the water supply for the consumers by improving the existing service level and objectionable water quality subject to advanced structural integrity and economical operation strategies of the pipe network. The study of the pilot project should be an example to solve the problems of the water supply utilities of eastern Germany which exhibited similar technical deficiencies. Financial aid for the study was partly supported by Federal German Foundation Environment (DBU) for the time interval of 1994–1998.

After verification of the pressure zones with rough demand data, the nodal water demands were estimated by using the amount of the annually metered residential water consumption and monthly commercial water use. The nodal demands were obtained by considering weighted distribution of the streetwise billed water consumption of the inhabitants. The commercial water usage was located to the nearest node of the pipe network. The simulation of some loading condition was performed depending on the nodal demand resulted from the distributed annual average water consumption to get roughly estimates for residual head distribution overall the pipe network. The nodes for field measurements were determined by analysing the simulation data. The measurement program was carried out on Nov. 3, 1994, in which inflows of the pumping and booster stations were metered and the water levels of the elevated reservoirs were recorded. Heads were continuously observed on 22 pre-specified critical nodes representing the individual pressure zones (see Fig. 8) and stored digitally by head measuring loggers over 24 h for the whole distribution system.

The analysis of observed data showed that the pressure zones exhibited unusual demand curves, the difference between peak and off-peak consumption was negligible in high pressure zones, where the

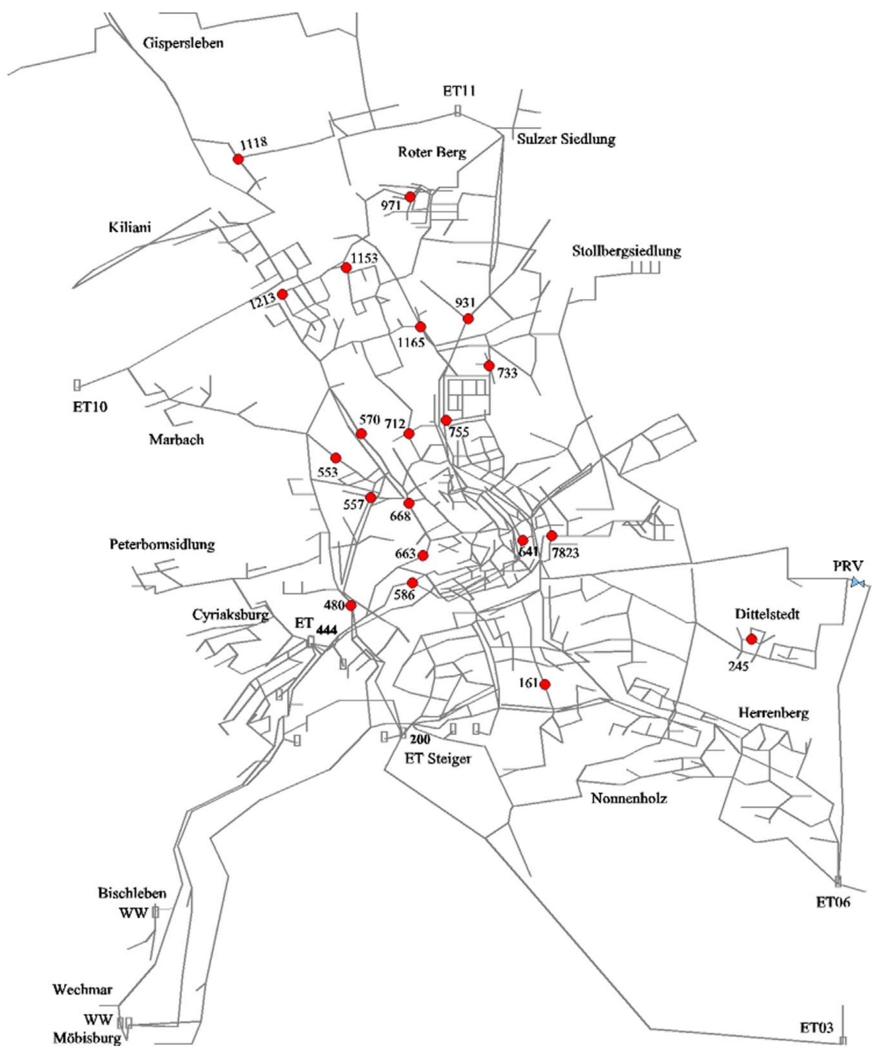


**Fig. 8.** Nodes of head measurements in pressure zones of WDN of the city of Erfurt in 1998.

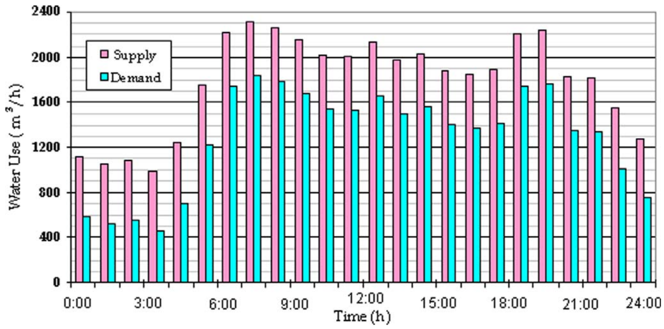


**Fig. 9.** Hourly water balance after interpolation of the leakage in the distribution network.

reservoir outflows of lower pressure zones were insignificant by off-peak demand in the night. Additionally, nodal heads were remarkably lower than a reasonable water demand situation in the northern part of HPZ Steiger West. As a result, a leak was detected in that part of the pressure zone where the crack was at the bottom-side (downward into the ground) of the pipe in the vicinity of the stream running through the city. It was difficult to observe the leakage as a pipe break, since the leakage was infiltrated into the gravel of the ground and passed through the stream bank in about 2 m depth. Temporal change of the leakage was interpolated to be about 50% of the inflows by minimum water demand in the night and about 22% by the peak demand as shown in Fig. 9. The sum of inflows into the network was 42,898 $m^3$/d

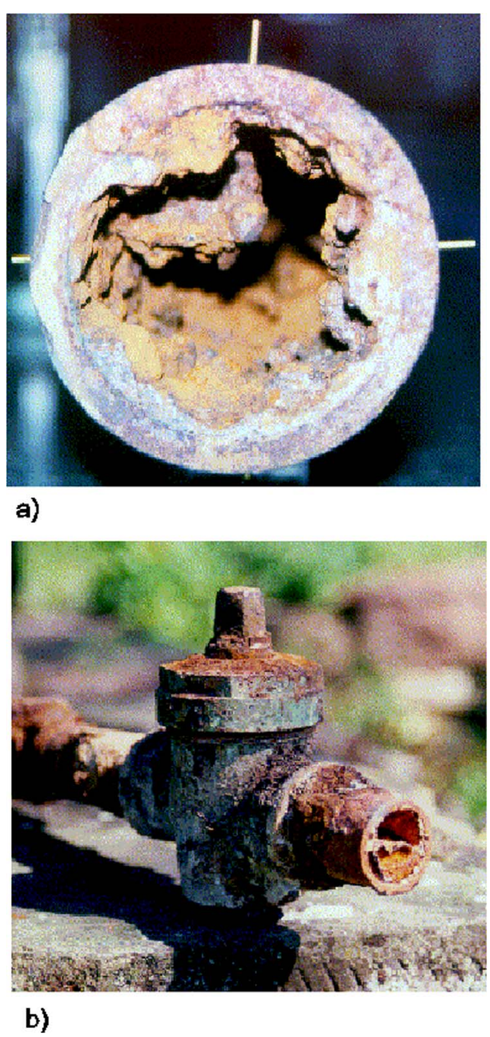


**Fig. 10.** Cross-section of an old pipe and household connection pipe, Erfurt.

on the day of the field measurement. It was about 144% of the daily average (30,347 $m^3$/d) of the year 1993 which was the basis for the estimates of nodal demands, where water utility recorded about $20.13\times0^6$ $m^3$/year inflows over the year 1993, i.e. 55,145 $m^3$/d were supplied into the network as a daily average.

The leakage from the pipe crack was detected after closely specifying the border of small supply district in the western part of high pressure zone. The amount of water being lost from the pipe crack was about 25% of the annual water supply, the remaining 20% water losses were proportionally added to nodal demand by the simulation data. Some old pipes from the city centre of the network were experimentally connected to the WDN in the hydraulic laboratory at the university of Karlsruhe to measure the head losses along the pipe segments by different flow rates under various heads [17]. The empirical head loss formula was verified to be represented in the form ($h_j=f_j\cdot q^\varepsilon$) by considering three different pipe sizes from the old region of the network in the city centre. The formula reveals that the absolute pipe roughness is to be between 15–30 mm, i.e. the pipes were very rough as shown in Fig. 10, in which the commonly used formula could not reflect the real hydraulics prevailing in pipe network.

The impact of the leakage on the storage capacity of ET Steiger is illustrated in Fig. 11 that includes the EPS data for 24 h in comparison with the observed water level in the most important two elevated tanks (Steiger and Cyriaksburg).

The separation of the pressure zones was simulated by considering the half of 74 isolation valves as TCV with smaller diameter size and higher valve coefficients since it was validated that the loosely closed valves allowed significant discharge into the lower pressure zones in order to obtain a reasonable mass balance between the pressure zones compared with the metered inflows from the reservoirs and the elevated storage facilities. Such a complex WDN could be efficiently simulated using former version of the presented numerical approach that took about 70 s on a PC for a specific demand pattern giving the

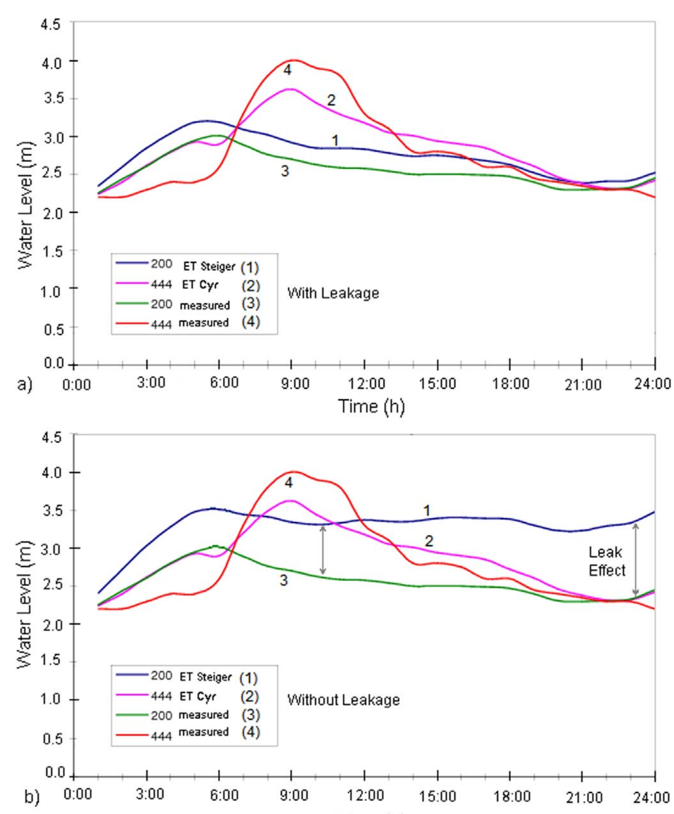


**Fig. 11.** Water levels of two elevated tanks after the observed and simulated values for 24 h.

measured inflows at the nodes of water works and pumping stations instead of pump characteristic curves. Additionally, the simulation model had to consider the booster stations switched by dispatcher (as TDV simulated) and not automatically using pressure level of a reference node. Moreover, an interesting observation could be reported that the EPS for 24 hourly time steps took about 25 min, where the execution of the first version of Epanet for the same dataset needed more than 90 min to converge to comparable results, since a sorting algorithm is still not implemented by Epanet2 to label the static closed pipes separating many pressure zones [87].

## 8. Concluding remarks

Pressure and flow regulating facilities are necessary for many large scale WDN to modulate either upstream or downstream pressure of the hydraulic systems. Additional to the different operating states of the control elements, the networks may have many closed pipes to improve the supply reliability and re-organise different pressure zones. The proposed numerical algorithm can directly include the zero flow boundary constraints of the closed pipes into the simulation model of network hydraulics without any adjustments in the network topological structure. Further, the algorithm can accommodate all types of closed pipe configurations by allowing a small reverse flow rates ($\varepsilon_{\min}$, $\varepsilon_{\max}$) and introducing *unknown head loss function* (UHF) for head losses in the related pipes so far as the provided system graph is topologically interconnected.

The simulation models are intensively used within the decision making process for the cost savings in the development of efficient and transparent pump scheduling policies and upgrading and maintenance strategies for existing WDN. The development and implementation of a robust and efficient numerical solution algorithm is of importance and unavoidable for comprehensive analysis of complex pipe networks. The numerical algorithm is modified to include the further control valves for simulating more complex large scale WDN. It was demonstrated that the numerical solution algorithm is robust and efficient and assured to converge in a speedy manner that is supported by providing a practical solid initial flow distribution and performing a local flow correction procedure for the loops involving closed pipes. The approach will significantly improve the computational efficiency of hydraulic simulation models of WDN. For further improvement, the algorithm possesses the capability to extend for applications on some complex configurations of pipe networks including many successive control valves, where it was not widely considered in the available program codes.

The simulation of the pipe network hydraulics including pressure/flow control devices is subject to more numerical difficulties in relation to the simulation of simple WDN, since the control facilities have dynamic operating states that have to be considered in the program code of the simulation model. However, the development and calibration of a hydraulic simulation model is a tedious task that involves many steps and adequate data collection and field measurements related to the distribution networks, their pressure devices/appurtenances [76,96], and appropriate computer program that can include the natural network complexity and deliver reliable and substantial results. The deficiencies of WDN can be fairly estimated by comparing the field measurements of pressure and flow rates with the simulated results of the model, i.e. insufficient pressure levels and capacity of storage facilities, leakage and water quality in pipe networks as well as the hydraulic behaviour of pressure regulating facilities such as booster stations (BOS), pressure regulating (PRV, PFV) and flow regulating (FCV, PSV) facilities.

The computational complexity of the pipe network simulation model is directly correlated with the number of control devices and their topological configuration, because the operating states of the downstream control valves are often dependent on the operating states of upstream control devices. Therefore, the network equilibrium is to be iteratively determined by considering the variable operating states of the control devices and the other system parameters that are definitely set as known such as the pipe characteristics (length, diameter size, roughness/friction coefficient), demand pattern at nodes, and heads of fixed nodes as well as the pump curves and suction head at the service reservoirs.

Results of computer runs for example networks showed that the iteration steps and the distribution of flows and heads were identical for head losses of PRV with several different initial values and various pseudo-loops. The iteration procedure could consistently converge in reasonable steps for individual operating states of PRV, though it revealed reverse flow rates in the first four iterations for pseudo loops of PRV. The solution algorithm could present different iteration steps depending on pseudo-loops with changing datum node. Consequently, the developed solution approach is robust and efficiently converge almost to the exact records with different positions of PRV which need different handling technique to incorporate into the simulation model, whereas other unidirectional devices will be represented with their addition head loss/input in the related pipes. Unfortunately, the advantage of the closed pipe approach for unidirectional control devices could not be exploited by pseudo-loops with PRV, since the pipe of the valve was not directly involved in the loops of the solution matrix. Further, the developed approach has to reconstruct the loop equations by Jacobian matrix for inactive and shutdown cases of PRV that can be considered as a pitfall. Improvement of the algorithm can be achieved by integrating some appropriate developments of the recently intensified studies related to PRV in loop equations.

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