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Review

Leakages in Water Distribution Networks: Estimation Methods, Influential Factors, and Mitigation Strategies—A Comprehensive Review

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Abstract: While only a minimal fraction of global water resources is accessible for drinking water production, their uneven distribution combined with the climate crisis impacts leads to challenges in water availability. Leakage in water distribution networks compounds these issues, resulting in significant economic losses and environmental risks. A coherent review of (a) the most widely applied water loss estimation techniques, (b) factors influencing them, and (c) strategies for their resilient reduction provides a comprehensive understanding of the current state of knowledge and practices in leakage management. This work aims towards covering the most important leakage estimation methodologies, while also unveiling the factors that critically affect them, both internally and externally. Finally, a thorough discussion is provided regarding the current state-of-the-art technics for leakage reduction at the municipal-wide level.

Keywords: water losses; leakage management; leakage; water networks partitioning; water distribution networks; water balance; minimum night flow

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1. Introduction

Water is an essential natural resource, crucial in sustaining human life as well as facilitating various societal endeavors, being utilized in industrial, agricultural, and livestock sectors, functioning as a fundamental component for the development and improvement of contemporary societies. As a result, the emergence of major civilizations throughout history is often linked to the advantageous presence of safe and easily accessible water sources [1–3].

However, according to Singh [4], a mere 0.7% of water resources can be found in rivers, lakes, and underground aquifers, thereby providing the sole viable sources for drinking water production. Furthermore, the uneven spatial and temporal distribution of precipitation, coupled with the escalating need for potable water resulting from population expansion and conflicting demands, renders water supplies limited in nature.

In addition, the shifts and trends of rainfall and evapotranspiration, in both time and space, a direct result of the climate crisis, will further affect the accessibility to surface and groundwater resources, as discussed in studies conducted by Bear et al. [5], Kirby et al. [6], and Garner et al. [7]. More specifically, a heavy precipitation event results in higher surface runoff rates, increased flood risk, and reduced recharge rates of groundwater aquifers [8]. Moreover, the global warming phenomenon contributes to the evapotranspiration's intensification, thus leading to increased irrigation water demands, which is the primary consumer of water on a global scale, as evidenced by studies such as those of Wang et al. [9] and Zhang et al. [10]. Consequently, there is an urgent environmental and societal need to implement efficient management practices to water distribution networks (WDNs), which constitute the core infrastructure for drinking water supply to users.

Water networks, irrespective of their age and construction materials, commonly exhibit leakages that significantly affect both the availability of water resources and the financial viability of the water supply and wastewater utilities, as the lost water remains un-capitalized. This raises their operational and managing costs while reducing their net-revenue (see, e.g., [11–15]). In addition, high leakage levels are indicative of potential deterioration of the quality of the water distributed to users, primarily due to possible undesirable inflows to the network when its pressure is low and/or during flow disruptions [16,17].

Water leakages in water distribution networks, beyond the profound wastage of a precious resource, present substantial environmental challenges ranging from damaging fragile ecosystems to further degrading areas facing water scarcity issues, while amplifying the pressure especially on agriculturally based communities. The increased stress intensifies the burden of already depleting water supplies, heightening the likelihood of water shortages and contamination.

The observed volume of leakages varies throughout different nations and water distribution networks (WDNs), which is primarily driven by factors such as the overall condition of the network and the maintenance and monitoring capabilities of the associated water authorities [18,19]. In cases where WDNs suffer from obsolescence and insufficient maintenance, the leakage rates can escalate to levels as significant as 70–80% of the system input volume (SIV), i.e., the total volume of water entering the network in a given time period (see, e.g., [14,20,21]). On the contrary, within meticulously maintained and monitored networks, the lost water volume is considerably diminished, amounting to a mere 7% of the total volume that enters the network [22]. On a global scale, it has been estimated that in 2018, the amount of non-revenue water (NRW), encompassing both leakages and unbilled authorized consumption (UAC), was on the order of 126 billion cubic meters annually, corresponding to approximately USD 39 billion [23].

Elimination of leakages, apart from being not technically possible due to their nature [19,24,25], it is also not cost-effective due to diminishing returns (i.e., the more the investment on leakage reduction, the less the additional benefit; see, e.g., [26–29]). Therefore, water supply agencies seek to determine the economic level of leakages, below which any further investment is not cost-effective (see, e.g., [30–33]), while applying the appropriate leakage reduction strategies proposed in the international literature.

Following the discussion above, the current work serves as a comprehensive literature review of the pervasive issue of water leakages within water distribution networks, both from an environmental and sustainability perspective but also from an economic and infrastructural standpoint. Furthermore, we aim towards investigating the infrastructure weaknesses leading to leaks and pipeline breaks, while highlighting the most efficient leakage control practices. Overall, the current article serves as an important resource for policymakers, water utilities, researchers, and stakeholders involved in water management, offering insights on water leakage identification and minimization, while also guiding future action and interventions.

Section 2 presents the most common water losses estimation methods and techniques by delving into the current water flow analysis, pressure monitoring, and data-driven modeling. Section 3 investigates the multifaceted factors influencing the occurrence and severity of leaks within distribution networks, while Section 4 discusses an array of strategies and techniques aimed at minimizing water leakages, providing insights into the diverse approaches utilized by researchers and practitioners in this field. Conclusions and future research directions are summarized in Section 5.

2. Water Losses Estimation Methodologies

The following three subsections provide a brief description of the three most used approaches for water loss estimation in water distribution networks (WDNs). The first two approaches are the water balance, also known as the top-down approach, and the burst and background estimates (BABE) approach, both introduced by the International Water Association (IWA; see [34] and [18], respectively). The third approach is the bottom-up approach based on the estimation of minimum night flow (MNF). The latter is decomposed into real losses (RL) and users' night consumption (UNC). A comprehensive analysis of the procedures involved in the top-down, bottom-up, and BABE methodologies is presented in AL-Washali et al. [35].

2.1. Water Balance (Top-Down) Approach

The concept and methodology of water balance estimation at both WDN (system-wide) and DMA (district metered area) scales (see [36]) were developed and established by the Water Loss Task Force of the International Water Association. To implement this approach, it is necessary to determine the total system input volume (SIV) and the authorized consumption (AC), which may be either billed or unbilled. The billed authorized consumption (BAC) refers to the water consumption (metered or un-metered) by legal users/consumers, also known as revenue water (RW), that is invoiced by the water agency for financial profit. The RW volume is determined by summing individual consumption volumes in the case of metered consumption, or approximated (in case of unmetered consumption) based on the nature and special attributes of each individual customer. The billed authorized consumption may encompass water that is sold in regions beyond the spatial coverage of the water distribution network (WDN).

Estimation of the lost volume of water is conducted by deducting the unbilled authorized consumption (UAC; water spend on firefighting, flushing of pipeline mains and sewers, street cleaning, etc.), whether metered or unmetered, from the non-revenue water (NRW), as indicated in Equation (1). Calculation of NRW requires subtracting the billed authorized consumption (BAC) from the system input volume (SIV), as shown in Equation (2).

$$\mathrm{WL} = \mathrm{NRW} - \mathrm{UAC} \tag{1}$$

$$\mathrm{NRW} = \mathrm{SIV} - \mathrm{BAC} \tag{2}$$

Given that WL is equal to the sum of real losses (RL) and apparent losses (AL):

$$\mathrm{WL} = \mathrm{RL} + \mathrm{AL} \tag{3}$$

So as to estimate RL using Equation (3), it is necessary to determine the AL volume by considering its various components, such as unauthorized consumption (UC), systematic metering errors, and inaccurate estimates of billed users' consumptions. These components can be quantified using semi-empirical estimates found in the international literature [37,38].

In particular, in high-income nations, the volume of UC is typically estimated to be around 0.1% [37] or 0.25% [39] of the SIV. In low-income countries, the estimated UC volume is higher, around 10% of the BAC [40] or 10% of the non-revenue water (NRW) [41]. To estimate systematic metering errors at the inlets of PMAs and/or DMAs, flow measurements should be conducted following the guidelines provided by the meters' manufacturer [42,43]. Historical flow timeseries and accompanying billing data can be used to identify inaccuracies in estimation of billed users' consumptions.

Water agencies undertake the assessment of AL by relying on their expertise and the technical specifications of the data collection and transmission systems. Figure 1 illustrates a schematic representation of the water balance components that are taken into account in water distribution networks (WDNs), in accordance with the suggestions of the International Water Association (IWA) [18].

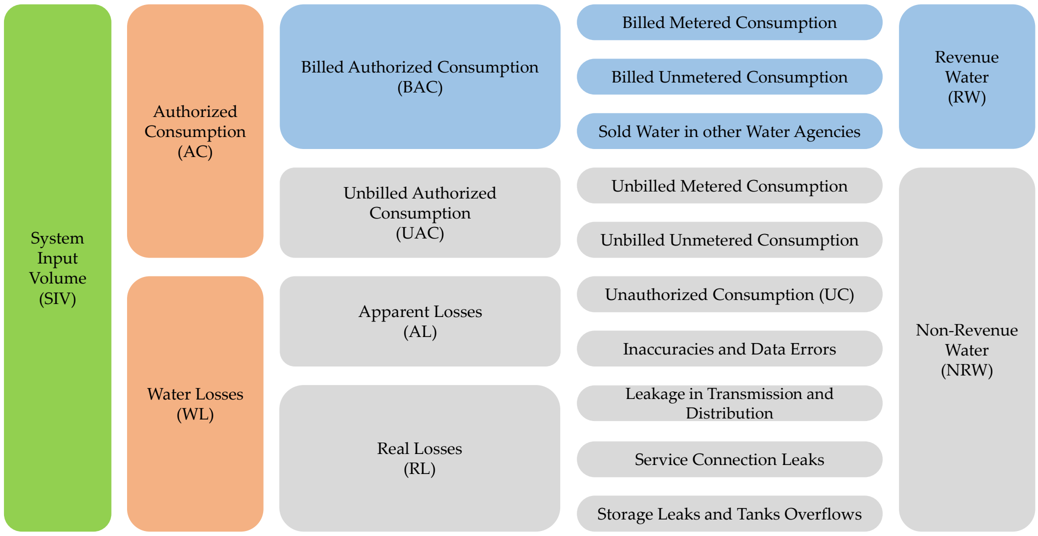

Figure 1. The components of water balance approach, as proposed by the International Water Association (IWA).

2.2. Burst and Background Estimates (BABE) Approach

The concept of burst and background estimates (BABE), introduced by Lambert in 1994 [34], attempts to objectively model leakage components, while not relying on empirical methods [39]. The BABE approach allows for the assessment of water losses (WL) components by estimating the volume of real losses (RL, i.e., leakages) and subtracting it from the overall volume of WL so as to estimate the apparent losses (AL). The fundamental idea of the BABE approach relies on the concept that RL are the sum of several instances of leakage. The magnitude of loss for each instance is determined by the average flow rate and the duration of each leak [44]. In the BABE concept [34], the volume of a single leak or burst is determined by multiplying the average flow rate by its duration, as indicated in Equation (4)

$$V = Q \times T \tag{4}$$

where V is the total volume of leakage, Q is the average leakage flow rate, and T is the duration of leakage. According to Lambert [34], a leak can be classified based on its flow rate to either (a) a burst with a high flow rate, which is easily identified and reported, or (b) a background leak with a low flow rate that cannot be detected by the service provider. The duration of a leak is determined by the water utility's policies, including the frequency of leakage detection procedures as well as the promptness of their response to repair detectable leaks. In this context, background leaks are consistently ongoing, while the length of time that reported bursts occur is influenced by the utilities' monitoring infrastructure and the applied water management strategies [39,45].

In order to determine the variables necessary for practical application of the BABE model, it is assumed that all bursts in excess of 500 L/h have been temporarily or permanently fixed. The initial model assumptions presented by Lambert [34] have been refined in subsequent publications by Lambert et al. [46], Lambert and McKenzie [47], and Lambert [48]. Ultimately, standard criteria were developed to streamline the estimation of the overall volume of leakages.

In order to accurately calculate real losses using BABE factors, it is necessary to estimate and combine both avoidable and unavoidable annual real losses. Lambert [48] reported the variables to be employed for estimation of the unavoidable (background) and avoidable (burst) (both reported and unreported) losses, based on the number of service connections and the length of mains. The outcome of the aforementioned estimation procedure is properly adjusted to reflect the average pressure of the entire water distribution network, as determined by the American Water Works Association [39]. Under this setting, the unavoidable annual real losses (UARL) may be calculated independently from Equation (5) [19,46]:

$$\mathrm{UARL} = (18\ \mathrm{L/km\ d^{-1}\ m^{-1}} \times L_m + 0.80\ \mathrm{L/conn\ d^{-1}\ m^{-1}} \times N_c + 25\ \mathrm{L/km\ d^{-1}\ m^{-1}} \times L_p) \times P \quad (5)$$

where UARL is measured in liters per day (L/d); L_m represents the length of the mains in kilometers; N_c accounts for the number of service connections; L_p is the total length of the house connection from the edge of the street to the customer meter in kilometers; and P corresponds to the average operating pressure of the network, measured in meters.

The sum of avoidable losses (i.e., estimated based on Equation (4) in the light of subjective judgment) and unavoidable losses (i.e., estimated based on Equation (5)) is used to assess the total volume of real losses. It is worth noting that the factors produced by the BABE model are user-friendly; however, it is imperative to verify the assumptions of the model prior to estimating WL, as it usually underestimates the leakage volume [35,49].

2.3. Minimum Night Flow (Bottom-Up) Approach

The predominant methodology for real losses (i.e., leakage) estimation in water distribution networks (WDNs) is the minimum night flow (MNF) technique, as elucidated by Liemberger and Farley [36], Hunaidi and Brothers [50], Thornton et al. [44], Tabesh et al. [51], Cheung et al. [52], Karadirek et al. [53], and Meseguer et al. [54]. The MNF estimation approach is based on the concept of the minimal human activity during late night and early morning hours, rendering MNF a reflective measure for leakage estimation as well as network's condition parametrization (see, e.g., [49,55–57]). Consequently, numerous research endeavors have focused on employing MNF analysis to assess background losses and the condition of WDNs with distinct characteristics.

AL-Washali et al. in 2019 [56] conducted a multivariate normal framework (MNF) analysis on an intermittent water distribution system in Zarqa, Jordan, utilizing 15 min resolution data during a 5 day period. Their findings revealed significant variability regarding the occurrence of flow minima (between 00:00 and 07:00 a.m.), influenced by the remaining volume of water in customers' tanks. Adlan et al. [58] investigated the frequency of minimum night flows, between 01:00 a.m. and 05:00 a.m., by analyzing flow data at 15 min intervals over a four-year period on 30 hydraulically isolated areas in the Kinta Valley region of Malaysia, indicating that 84.2% of MNF incidents occur between 02:15 and 04:15 a.m. Similarly, Verde et al. [59] identified the minimum night flow values between 01:40 and 03:30 a.m. using high-frequency (1 min resolution) flow data from a small pressure management area (PMA) in Lenola, Rome (Italy), while Muhammetoglu et al. [60] employed MNF analysis on flow data collected from Antalya (Turkey) at 15 min intervals, specifically between 00:00 and 05:00 a.m.

Bakogiannis and Tzamtzis [61], Hamilton and McKenzie [62], and Makaya [63] limited the time window for estimating the minimum night flow (MNF) to the period between 00:00 and 04:00 a.m., while Lee et al. (2005) examined a small pressure management area (PMA) in Korea, utilizing 1 h data within the same time frame, and categorized land uses into business and residential sectors. Expanding on this, Tabesh et al. [51] noted that the most significant reduction in nighttime water consumption occurred between 03:00 and 04:00 a.m., particularly among residential users.

Peters and Ben-Ephraim [64] conducted a nighttime analysis using 15 min resolution flow data from a district metered area (DMA) in Berbic (Guyana) over a 15 day period. In their study, minimum night flows estimated by averaging measurements between 02:00 and 04:00 a.m. yielded higher MNF estimates compared to deriving minima from the original time series. Farah and Shahrour [65] utilized a 15 min moving average window so as to extract MNF estimates from 02:00 to 05:00 a.m. over a 16 month period, using data from the Scientific Campus of the University of Lille in France from January 2015 to April 2016.

One can conclude that the process of scanning and extracting minimum flow values during the night may start or conclude at different hours depending on the study or application. Typically, seasonal consumption patterns and weekday effects are not considered, leading to inaccurate MNF estimates, as discussed by WSAA [66]. Similarly, disregarding the temporal resolution of the flow measurements may result in unrealistically low MNF estimates due to signal variability caused by various factors such as flow interruptions, pressure waves, equipment malfunctions, environmental conditions, and suspended solid concentration [42,62,67].

In a recent study, Serafeim et al. [68] developed two probabilistic methodologies for estimating MNF using statistical metrics, which lead to reliable estimates by considering typical nighttime flow conditions during periods of low demand throughout the year. The initial method involves determining a suitable scale for averaging nighttime flows over time, so as to mitigate noise effects in the resulting MNF estimates. The second approach is simpler and more intuitive, estimating MNF by averaging the lowest modal values observed during nighttime hours throughout the low consumption period of the year.

Upon estimating the minimum night flow (MNF) for a chosen pressure management area (PMA) or district metered area (DMA), it becomes necessary to decompose it into two components: the net night flow (NNF), representing the leakage rate during nighttime hours (as per Farley [20]), and the users' night consumption (UNC), encompassing both authorized and unauthorized consumption. This decomposition is achieved through the balance Equation (6):

$$\text{NNF} = \text{MNF} - \text{UNC} \tag{6}$$

UNC can be estimated under the assumption that approximately 6% of the residential population remains active during nocturnal hours, with an average consumption of 10 L of water per capita [62]. As for non-domestic consumers, their nighttime consumption is typically directly metered on-site.

Given that NNF represents the leakage rate during nocturnal hours, when network pressures are generally lower due to pressure management, the determination of real losses (RL) occurring throughout both day and night necessitates the multiplication of NNF by the night–day factor (NDF, [69]):

$$\text{RL} = \text{NNF} \times \text{NDF}, \text{NDF} = \sum_{i=0}^{24} \left(\frac{P_i}{P_{MNF}} \right)^{N_1} \tag{7}$$

where RL represents the real losses in cubic meters (m^3); NNF denotes the net night flow in cubic meters (m^3); P_i signifies the mean pressure during each hour i of the day; P_{MNF} denotes the mean night pressure during the period of MNF estimation; and N_1 corresponds to the leakage exponent, which varies from 0.5 for rigid pipes (such as steel, cast iron, plain concrete, reinforced concrete, vitrified clay, and asbestos cement) to 1.5 for flexible pipes (such as PE, PVC, HDPE, and FRP) [37,69–71], with an approximate average value of 1.15 [69,70].

2.4. Comparison

A weakness of the IWA's water balance (or top-down) approach is that it heavily relies on semi-empirical assumptions regarding the assessment of the unbilled authorized consumption (UAC) and apparent losses (AL), which may result in inaccurate estimates due to the increased level of uncertainty. Similarly, the burst and background estimates (BABE) approach is based on the assumption that all major bursts have already been fixed, while the unavoidable annual real losses (UARL) are estimated empirically using Equation (5). In this context, the probabilistic minimum night flow method should be preferred as more rigorous, when high-resolution flow timeseries are accessible and there is an adequate comprehension of the diurnal cycle of consumption patterns, while it does not require any empirical assumptions. Additionally, it allows for confidence interval estimation of flow minima, which are representative of leakage rates, improving usefulness of the method for water resources management.

3. Leakage Related Factors

The causes of cracks in water supply pipelines can be categorized into four subgroups: (1) structural factors, which are related to the physical properties of the pipelines; (2) external factors, which are linked to the surrounding environment; (3) internal factors, which involve the hydraulic characteristics of the flow inside the pipelines; and (4) factors specifically associated with the maintenance and repair of the pipeline network [12,55,72,73]. Human decision can be incorporated into the aforementioned classifications, both during design and construction as well as during operation of the network (such as pressure regulation, valid monitoring, maintenance, and repairs). In the following four sub-sections, the focus is on the structural, external, internal, and maintenance-related factors that contribute to the formation of cracks in water distribution networks. In addition, the specific mechanisms that underlie each subgroup of factors are highlighted, with emphasis on the interconnections within the wider framework of pipeline infrastructure management and maintenance.

3.1. Structural Factors and Physical Pipe Properties

Structural and physical factors include the pipeline's diameter, age, and material, as well as the number of connections per unit length of pipeline.

3.1.1. Diameter and Wall Thickness of the Pipeline

The material strength of pipelines, which affects their vulnerability to inherent operational and environmental hazards, is determined by their diameter and wall thickness. The tensile stress developed in the walls of a pipeline with length L and diameter d is expressed through Equation (8)

$$\sigma_w = \frac{\Delta P \cdot d}{2 \cdot t_w} \tag{8}$$

where σ_w is the developed tension, t_w is the pipes' wall thickness, and

$$\Delta P = P_{int} - P_{ex} \tag{9}$$

where P_{int} is the internal (i.e., inside the pipe) pressure and P_{ex} is the external pressure based on the environmental conditions.

Although Equation (8) indicates that the developed tensile stress in a pipe is directly proportional to its diameter (d) and inversely proportional to its wall thickness (t_w), several studies, including those of Kettler and Goutler [74], Andreou et al. [75], Kanakoudis and Tolikas [76], Christodoulou et al. [77], and Mutikanga et al. [40], mention that the occurrence of pipeline failures is inversely proportional to the diameter. In a recent work, Langousis and Fourniotis [78] studied the aforementioned correlation by analyzing the internal diameters (d) and wall thicknesses (t_w) of commercial HDPE and PVC pipes in the 16 atm class and highlighted that the ratio $\lambda = d/t_w$ remains approximately constant, regardless the internal diameter of the pipeline, as long as the material remains the same. For a detailed discussion on the effect of internal pressure on leakages, the reader is referred to Section 3.3.2 below.

Hence, the factors that contribute to a rise in the occurrence of failures when the pipes' nominal diameter decreases should be attributed to second-order causes. For example, connecting pipes with small diameters poses greater challenges, resulting in higher rates of failure of the corresponding joints. Furthermore, it is worth noting that the moment of inertia of a circular pipeline is directly proportional to the fourth power of its diameter. As a result, pipelines with larger diameters are more resistant to mechanical stresses caused by differential displacements due to soil settling, as well as dynamic loads imposed by earthquakes and moving vehicles [79].

3.1.2. Pipeline Material and Aging

The most widely used materials for constructing water mains include polyvinyl chloride (PVC), high-density polyethylene (HDPE), steel, and asbestos cement. Asbestos-cement pipes (A/C), which are no longer utilized in the construction of new water supply networks, exhibit greater rates of failure mostly due to their aging. Following steel pipes and PVC pipes, HDPE pipes exhibit the lowest failure rate, as evidenced by Kettler and Goutler [74], Sharafodin et al., [80], Goutler and Kazemi [81], and Kanakoudis and Tolikas [76].

Oxidation of steel pipelines over time leads to a decrease in their cross-sectional area and strength, making them susceptible to variations in internal pressure and external loading. Common practice suggests the use of steel pipes when nominal diameters surpass 630 mm and/or pressures exceed 32 atm. Such requirements typically align with significant water supply undertakings, where precise estimation of pipeline replacement intervals becomes particularly crucial to minimize risk of failure [78].

International literature states that the lifespan of steel pipelines typically falls between 40 to 100 years, as shown by Hoye [82], Walkski and Pelliccia [83], and Pantokratoras [84]. It is worth mentioning that the aforementioned timespan refers to networks where (a) pipelines are positioned and connected without defects, (b) operating conditions are suitable and pipelines are protected from external factors (e.g., cathodic protection in corrosive soils), and (c) the network is maintained as per its construction specifications [78].

Based on the aforementioned considerations, various models have been proposed to parameterize steel pipeline failures as a function of their operating time. This endeavor aims towards fostering a more economically viable operation and management of water supply networks, achieved through a comprehensive program encompassing pipeline maintenance and replacement. Furthermore, Shamir and Howard [85] studied both a linear and an exponential model for the parameterization of fractures in steel pipelines, concluding that the second one most accurately describes the increase in vulnerability of steel pipelines as a function of time.

Clark et al. [86] developed a linear multiparameter model to estimate the interval between network establishment and the first failure, as well as an exponential multiparameter model for subsequent occasions. Following the results of Clark et al. [86], Andreou et al. [75] suggested bifurcating the lifetime of water supply networks into two distinct phases: an initial period characterized by fewer than three fractures occurring over several years, and a subsequent period following the initial phase. During the initial period, risk assessment models based on proportionality assumptions were recommended, while Poisson models were proposed for the subsequent phase. A similar approach was proposed by Goutler and Kazemi [81], applying non-homogeneous Poisson-type models to represent successive fractures subsequent to the initial failure event.

Regarding PVC pipelines, Folkman [87] stated that their durability remains unaltered even up to a century under optimal installation and network operation conditions, due to their resistance to oxidation and slow change of the corresponding material properties over time. Numerous instances in international literature have examined the mechanical properties of PVC pipes in different installed systems. In 1985, Lankashire [88] conducted tests on pipelines ranging from 4 to 16 years old and determined that aging did not influence the occurrence of fractures. Similarly, Alferink et al. [89] examined 37-year-old pipelines, noting no significant alteration in mechanical properties compared to those at installation. Stahmer and Whittle [90] reached comparable findings to the two prior investigations, with 30-year-old pipes affirming the sustained expected strength without any reduction. Moreover, Stahmer and Whittle [90] and Cakmakci et al. [91] noted that issues in PVC water supply systems often arise early due to inadequate design and flaws in pipe installation and connection. Similarly to PVC, HDPE pipes exhibit minimal oxidation and gradual deterioration of physical properties over time.

Similarly to the aforementioned studies, Serafeim et al. [92] observed through the development of a probabilistic framework for the parametric modeling of leakages in water distribution networks that the aging process had minimal impact on the leakage rates in the network, which consisted of 86 distinct pressure management areas (PMAs), comprising PVC and HDPE piping infrastructure.

3.1.3. Connections per Unit Length of Pipeline Grid

Improper installation technics (i.e., insufficient pipe sealing or welding) can result in prone to failure (i.e., weak) pipe-joints. Furthermore, material compatibility is often disregarded, leading to uneven thermal expansion and contraction, resulting in rapid material corrosion. This effect can be amplified in areas where frequent changes in flow rates or pressure occur, such as near pumps or valves, or in cases of important external stresses, such as ground settlement or movement of the surrounding soil.

Many studies have shown that a significant portion of leakages can be attributed to flaws in the connections between pipelines, which corresponds to more than 50% of the total water losses, even in cases when networks exhibit low link density [44,46,93,94].

In their study, Serafeim et al. [92] showed that the leakage rates in a water distribution network are directly related to the overall density of connections and junctions along the pipeline grid, which embodies (a) the density of connections on the main (comprising the number of individual users' connections on the pipeline grid and the number of hydrometers); (b) the density of valves; and (c) the density of nodes introduced at pipe junctions, fire hydrant locations, and locations where changes of pipe diameter and/or material occur.

3.2. Environmental Factors and Physical Pipe Properties

External and environmental effects on pipelines encompass a wide range of factors, beyond the pipeline structure itself. Among them, soil condition is crucial, as variations in soil composition, moisture levels, and stability can impact the integrity of the pipeline over time. Additionally, temperature fluctuations can lead to thermal expansion or contraction, potentially affecting the pipeline's material properties as well as its structural stability. Furthermore, external loads, such as those induced by wheeled traffic or other mechanical forces, present additional challenges.

3.2.1. Soil Condition

Shielding the pipeline from external factors is crucial and, therefore, water supply pipelines are often installed in a trench with a minimum of one meter of soil coverage to protect them from temperature fluctuations, sunlight, and traffic.

Soil subsidence caused by external factors is a common phenomenon with significant implications for the pipeline integrity. Displacements resulting from soil subsidence can trigger the loosening of the compacted gravel inside the trench where the pipeline lies on, resulting in increased tensile stresses in its walls [95]. Furthermore, soil relaxation renders pipelines vulnerable to external loads posed by natural occurrences (such as falling stones) and human activity (such as car traffic and road maintenance). Unstable soil may also trigger the formation of cracks and holes along the pipeline, as angular and/or sharp fragments under the influence of external loads may impose significant point pressure on the pipes' walls, leading to crack formation [78].

Surface erosion of soil poses a significant risk to pipelines by exposing them to various environmental factors capable of altering construction materials and compromising their strength [80]. Examples include PVC and HDPE pipelines affected by sun radiation and steel exposed to moisture and atmospheric air and/or corrosive salts found near the trench. The presence of corrosive salts can accelerate the corrosion process, leading to metal degradation and potential water quality issues. Corroded metal can impact water quality and flavor and, at high levels, pose a public health hazard [84]. Eisenbeis et al. [96] classified soil types into three categories: (a) very aggressive, including tidal zones, natural soil with resistivity under 750 ohm·cm and/or pH less than 5, polluted soils, and stray current; (b) moderately aggressive, including clay areas, wetlands, and inhomogeneous soils; and (c) not aggressive, with resistivity over 2500 ohm·cm or dry soils (i.e., sand) and/or moraine terrains.

The level of the groundwater table can have a significant impact on the quality of drinking water. In particular, if the level of the groundwater table exceeds the bearing height of the pipelines, the network becomes vulnerable to parasitic inflows from existing connections and possible cracking, especially in the event of a water supply interruption. This phenomenon is more pronounced if the graded fill material of the trench is loose, and significantly more dangerous if the aquifer is subject to biological and/or chemical pollution. In the latter case, the rate of deterioration of the pipe walls is also affected by the corrosive properties of the contaminant [78].

3.2.2. Temperature

Temperature changes impose variations to the length of pipelines, which may stress pipe connections and potentially disrupt the compaction of the soil material surrounding the pipeline. The change in length (ΔL) of a pipeline due to temperature variation can be quantified using the relationship:

$$\Delta L = L \times \alpha \times \Delta T \tag{10}$$

where ΔL is the change in length of the pipeline (in mm), L denotes its initial length (in m), α signifies the coefficient of thermal expansion (in mm/m/°C; see Table 1), and ΔT corresponds to the temperature change in °C.

Table 1. Coefficients of thermal expansion of water pipe materials (corresponding to average values given by manufacturers; see e.g., [78]).

Pipe Material	α (mm/m/°C)
PVC	0.200
HDPE	0.180
Steel	0.012

Figure 2 illustrates the change in length of a 100 m long pipeline as a function of the temperature variation, ΔT, for different materials. One sees that PVC and HDPE pipes are more sensitive to temperature changes, which also affects their mechanical properties. More specifically, increasing temperatures have a detrimental impact on the mechanical strength of PVC and HDPE pipes, as stated by Al-Hashem and Al-Naeem [97]. Furthermore, sub-zero temperatures can cause pipeline bursts due to ice development inside the pipes. As soil temperature has a significantly smaller range of variation than atmospheric air, ensuring that the pipeline is adequately covered by a soil layer of at least 1 m thickness is crucial (see, e.g., [98,99]).

3.2.3. External Loads

Eisenbeis et al. [96] studied the impact of traffic load on pipes positioned beneath roadways, categorizing the traffic load into six distinct groups: (a) low traffic (<25 trucks/day), (b) high traffic (25–300 trucks/day), (c) extremely heavy traffic (>300 trucks/day), (d) pedestrian road, (e) subsidiary road, and (f) main road. Their study indicated a noteworthy correlation between the frequency of pipeline failures and the augmentation in traffic volume. It was observed that, as a general trend, increasing the depth of pipeline placement leads to a reduction of the stresses imposed to the pipeline by external loads, consequently reducing the likelihood of failure.

Furthermore, Eisenbeis et al. [96] studied the combined effect of traffic load and pipe's location, indicating that a reduction of the uncertainty of the latter would significantly increase the accuracy and reliability of the associated failure models, while also providing cost-effective solutions.

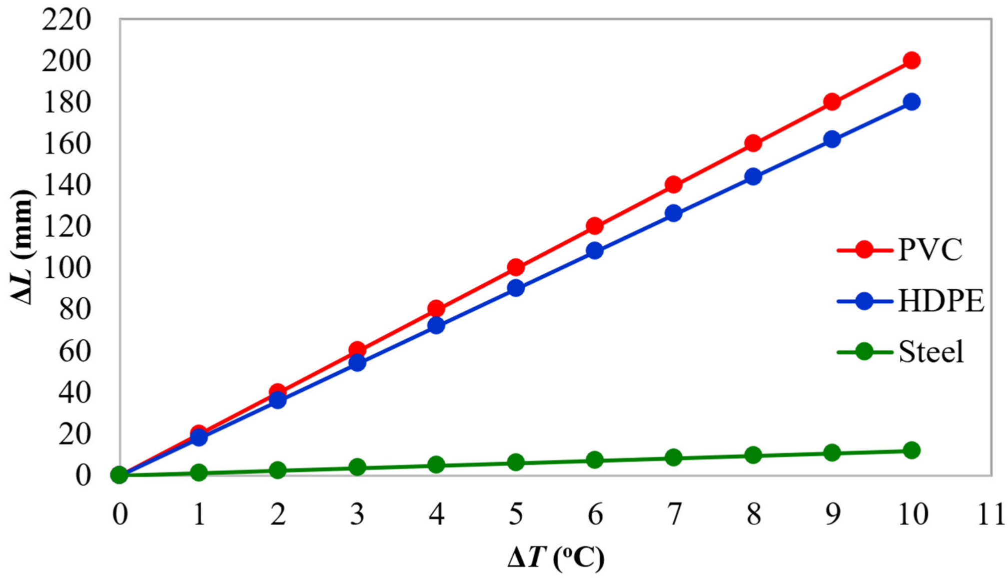

Figure 2. Change in length of a 100 m long pipeline utilizing PVC (red line), HDPE (blue line), and steel (green line) as a function of temperature change.

Despite the significance of the aforementioned findings, it should be noted that a gap exists in the international literature regarding the optimal installation depth of water mains, especially when taking into account the varying types and intensities of external stresses. Addressing this research gap would not only enhance the understanding of pipeline performance under a diverse set of traffic conditions, but also facilitate the development of more effective infrastructure design and management strategies globally.

3.3. Internal (Hydraulic) Factors

Internal or hydraulic factors, mainly velocity and pressure, significantly influence leakage rates within a water distribution system. High velocity can increase leaks by exerting increased force on vulnerable points or accelerating erosion rates over time, while, according to Torricelli's law, high-pressure differences intensify leakage rates.

3.3.1. Velocity

Excessive water velocities typically damage the inner walls of pipelines and harm pipe connections. In addition, overpressures and underpressures resulting from hydraulic shocks caused by the abrupt closure of valves are directly proportional to the water velocity in the pipeline. Equation (11) describes the overpressure ΔP (in m) resulting from the sudden closure of a valve:

$$\Delta P = \rho \times \alpha \times \Delta v \tag{11}$$

where ρ is the density of water (in kgr/m^3), α is the velocity of pressure waves in water ($\approx 10^3$ m/s, see, e.g., [100]), and Δv is the change in water velocity during the sudden valve closure. Avallone [101] noted that for every 1 m/s change in velocity, the pressure increases by approximately 10 atm (10^6 Pa). Hence, strict adherence to velocity limitations is imperative to ensure the smooth and uninterrupted operation of the water distribution network.

3.3.2. Pressure

According to Torricelli's law, the velocity v that water flows out from an orifice/crack of surface area A located along a pipeline is directly proportional to the square root of the pressure head h (in m) in the pipeline:

$$v \propto h^{0.5} \rightarrow Q = v \times A \propto h^{0.5} \tag{12}$$

Consequently, an increase in the operational pressure of a water supply network inevitably leads to escalation of real water losses [92,102]. Lambert [103] proposed a general formula (see Equation (13)) to describe the linkage between real losses and pressure in water distribution networks:

$$Q = c \times h^{N1} \tag{13}$$

where c denotes the flow coefficient and $N1$ is a loss factor that ranges from 0.5 to 2.79, as reported by Lambert [103,104] and Farley and Trow [105]. The ratio in Equation (14) allows for direct estimation of the changes imposed to leakage rates by pressure variations in the network:

$$Q_1/Q_2 = (h_1/h_2)^{N1} \tag{14}$$

Figure 3 illustrates the relationship between the ratio of leakage rates Q_1/Q_2 and the ratio of the pressure heads h_1/h_2 for different values of the loss factor $N1$. It is evident that the estimated losses are notably sensitive to the chosen value of the loss factor $N1$, which is determined empirically based on pipeline material, soil conditions, crack morphology, and the overall demand on the network, as discussed by van Zyl and Clayton [106] and van Zyl [107].

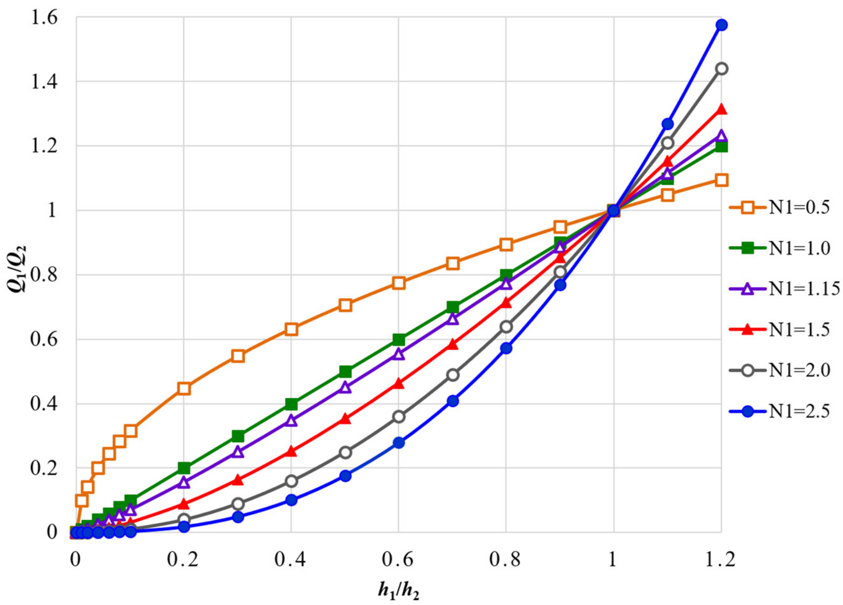

Figure 3. Ratio of leakage rates Q_1/Q_2 as a function of the ratio of pressure heads h_1/h_2 for different values of the loss factor $N1$.

Cassa et al. [108] studied the relationship between pipeline material and leakage rates using the finite element method, concluding that there is an approximately linear relationship between pressure and water losses. Similar results have also been obtained by Ferrante [109], while also considering potential plastic deformations of the materials.

In a recent study, Serafeim et al. [72,110–115] studied the minimum night flow (MNF) in pressure management areas (PMAs), revealing that that in PMAs with considerable leakage levels, the MNF estimates increase almost linearly with increasing inlet pressure.

As shown in Equation (12), the outflow Q is directly proportional to the cross-sectional area A of the orifice (or crack) from which the outflow occurs, significantly affecting the lost volume of water. Additionally, the geometric characteristics of the outlet cross-section are equally crucial, as highlighted in the studies of Casa et al. [108] and De Marchis and Milici [116], which influence the coefficient in Equation (13).

3.4. Design, Maintenance, and Repair of the Pipeline Network

The continuous and uninterrupted maintenance of a water supply network is an important factor when it comes to increasing its lifetime and reducing real water losses. Human involvement is present throughout the lifetime of a water supply network, from design and construction to operation and management. Of particular importance is the designer's knowledge and experience of domestic and international water supply network design and leakage control practices, as well as a good knowledge of the study area and its water requirements.

With regard to the construction and maintenance of the water supply network, the personnel involved must be fully trained and qualified, as the majority of failures are due to poor workmanship (e.g., defects in connections, inadequate positioning of pipes within the trench). A typical case is that of Iran [95], where 1,000,000 new leaks were identified in 1998 in the connections of the network's pipelines. Finally, pipeline injuries can be caused by the work of third-party crews, such as electricity, gas, and telephone, whose networks are located near water pipelines.

More details regarding the repair of water distribution networks are presented in Section 4.1.

4. Leakage Reduction Technics

While water losses constitute an increasing stress for water related authorities, resulting in revenue losses as well as significant environmental degradation, their efficient management is essential towards ensuring both their sustainability and reliability. In addressing this issue, strategies focusing on pipe repairs and replacement have long been employed to mitigate leakage rates and effectively enhance network performance. Additionally, advancements in hydraulic as well as computer engineering have led to the development of innovative methods, such as network partitioning into district metered and/or pressure management areas (DMAs and PMAs, respectively), so as to optimize pressure control and reduce leakages. This chapter presents a comprehensive review of the aforementioned approaches, delving into the principles of pipe repair and replacement, as well as the various implementation approaches for water distribution networks partitioning, aiming at enhanced pressure regulation.

4.1. Pipe Repair and Replacement

An important decision-making process arises when pipelines exhibit signs of structural and/or functional deterioration, requiring the consideration between repairing the damaged pipes or opting for excessive pipeline replacement. Repairing interventions typically target localized breaks, with minimal effect on the overall pipeline infrastructure due to their nature, whereas pipe replacement usually involves the complete substitution of the aged segments, thus offering a thorough restoration of the pipeline's integrity. While full pipeline restoration requires a significant financial investment, many studies have focused on determining the optimal pipe replacement strategies, in terms of benefit-to-cost ratio. Xu et al. [117] suggested the classification of the above studies into two categories: (a) those focusing on determining the optimal timing for pipe replacement, and (b) models prioritizing pipelines for replacement.

The studies found in the international literature regarding the first category aim towards fostering an economically viable operation and management of water supply networks, achieved through a comprehensive program including pipeline maintenance and replacement. As outlined by Shamir and Howard [85], critical factors for economic analysis include: (a) estimating breakages in existing pipelines, (b) assessing the average cost of water lost per breakage, (c) evaluating the expense related to breaks repairing, (d) determining the cost associated with pipeline replacement, (e) forecasting bursts in new pipelines, and (f) estimating the overall cost of network replacement.

Similar to Shamir and Howard [85], Kleiner et al. [118] considered both structural and functional deterioration so as to determine the pipelines' lifespan, while other works introduced multi-objective models to simultaneously address maintenance costs and the impact of deteriorating pipes on water pressure [119,120]. Following the aforementioned strategies, Park and Loganathan [121] introduced the concept of threshold break rate, indicating the economic viability of replacement upon exceeding this threshold, thereby highlighting the significance of forecasting the likelihood of pipe failures during replacement management planning [117].

The second category (i.e., models prioritizing pipelines for replacement) incorporates a plethora of evaluation criteria so as to prioritize pipes for replacement, focusing on the discerning allocation of resources designated for substitution initiatives. While focus lies on identifying the most vulnerable pipes, rather than determining the precise optimal timing for pipe replacement, a variety of models have been developed to assess these priorities, incorporating diverse performance indicators and methodological approaches [122–125]. Ho et al. [123] used a GIS-based artificial neural network to prioritize the order of pipe replacement, while De Oliveira et al. [122] proposed a density-based hierarchical clustering approach using network's infrastructure failure data, so as to define the associated pipe breakage indicators. Luong and Fujiwara [124] and Luong et al. [125] mainly focused on the optimal allocation of funds during the prioritization modeling, so as to maximize the net benefit while also considering the network's hydraulic reliability.

Evidently, the aforementioned categories complement each other to some extent: the first focuses on the timing of pipe replacements combined with economic efficiency considerations, while the second relies mostly on proactive measures determined by vulnerability assessment results. Selection between the two types of pipeline management strategies should be based on careful evaluation of the specific characteristics of the pipeline network, the available funds, and the associated risk tolerance levels. Ultimately, a combination of methodological elements from both categories of methods could result in the formulation of a comprehensive pipeline management plan, balancing long-term infrastructure resilience with economic considerations.

4.2. Partitioning of Water Dinstribution Networks

While it is not technically possible to completely eliminate leakages (see [19,24]), one effective method for reducing them is to divide the water distribution networks (WDN) into district metered or pressure management areas (DMAs and PMAs, respectively). This is followed by reducing the inlet pressures to the lowest acceptable limit that still meets the consumption/demand requirements [24,45,92,105,126–152].

The process of dividing WDNs into DMAs and PMAs was initially introduced in the United Kingdom during the 1980s [20,45]. It stands as one of the most widely used strategies for reducing leakage rates in WDNs, given that actual losses tend to increase with rising water pressure. Reports suggest that implementing network partitioning in the United Kingdom resulted in a noteworthy 85% reduction in water losses, as documented by Farley [20] and Kunkel [135]. However, the complexity of partitioning an existing WDN into individual DMAs (or PMAs) becomes evident when one considers the plethora of potential solutions and conflicting criteria, such as balancing between leakage reduction and hydraulic resilience (i.e., the ability of a network system to respond effectively to stress conditions; see [136]).

Numerous contemporary methodologies rely on semi-empirical standards, which involve establishing the boundaries of district metered areas (DMAs) or pressure managed areas (PMAs) based on various factors such as natural features (e.g., riverbanks), administrative divisions (e.g., districts), or engineered landmarks (e.g., roads). These considerations encompass the proximity to reservoirs (tanks), population density, and altimetry of individual DMAs [37,69,71,137,138]. While partitioning the network into smaller DMAs (or PMAs) may result in reduced leakage rates and expedited detection of critical events, it concurrently escalates both the delineation expenses (see [12,139,140]) and the overall hydraulic vulnerability of the study area, as indicated by low hydraulic resilience index values (see [141,142]). Typically, the optimal size of district metered areas

(DMAs) or pressure management areas (PMAs) is determined empirically, aiming to ensure that the number of service connections ranges between 500 and 5000 properties [44,143]. Karadirek et al. [53] have advocated for a maximum DMA/PMA size of approximately 1000 connections, suggesting an upper limit for optimal partitioning.

Recently published research indicates the utilization of heuristic methodologies in the partitioning of water networks, which typically consists of the clustering and sectorization phases [12,44,136,138–151]. During the first (i.e., clustering) phase, district metered areas (DMAs) are established based on the original connectivity and topology of the water distribution network, while the subsequent phase (i.e., sectorization) aims to optimize the placement of pressure regulation valves and meters to enhance network performance, thereby minimizing both the water losses and the associated financial cost [144]. Perelman et al. [153], Di Nardo et al. [154], and Khoa Bui et al. [144] have categorized the developed clustering procedures into six distinct groups, delineated by their algorithmic structures: (a) methods rooted in graph theory, (b) algorithms based on community structure, (c) modularity-based algorithms, (d) methods employing multilevel graph partitioning, (e) algorithms leveraging spectral graph theory, and (f) the multi-agent approach.

The graph theory algorithm stands out as the most common method for water network clustering, aiming at partitioning network nodes into a desired number of (ideally) equally sized clusters while minimizing the number of inter-connection edges between different clusters [153]. Notably, the depth-first and breadth-first searches are the prominent variants of this approach [145,150,155,156], using connectivity analysis to determine the number of independent groups, usually based on a shortest-path search while considering both pipe characteristics and mean nodal pressures as weighting factors [155,157–160]. While the computational demands of graph-theory-based partitioning algorithms often result in time-consuming applications, Karypis and Kumar [161] introduced the multilevel graph partitioning approach, which employs parallel computing strategies to distribute the computational load evenly across multiple processors or processor cores, thereby minimizing algorithm runtime [149,162–164].

The community structure algorithm adopts a bottom-up hierarchical strategy [153], aiming at maximizing partitioning effectiveness using the modularity quality function (see [165]) by combining sub-clusters to achieve the highest modularity until all computational nodes are grouped. Diao [148] first applied this approach to water network clustering, utilizing an oriented dendrogram cutting method to size clusters between 300 and 5000 properties. Subsequently, Campbell et al. [166] and Ciaponi et al. [167] employed similar approaches, excluding main transmission pipes from their analysis due to the potential extensive network-wide impact.

Spectral graph clustering, which relies on eigenvector and eigenvalue analyses of graph Laplacian matrices [154], incorporates both adjacency and weight matrices, with the latter considering hydraulic pipeline parameters. Notably, careful consideration of weight criteria selection is essential, as it significantly influences clustering outcomes [144,154]. Liu and Han [168] extended the spectral graph method to an automated DMA (or PMA) design approach, incorporating multicriteria decision methods to determine the optimal solution.

The multi-agent approach, introduced by Izquierdo et al. [169], conceptualizes WDN pipes and nodes as autonomous yet interactable agents. As a result, by leveraging agents' hydraulic characteristics, the network is partitioned into homogenous clusters by connecting nearby nodes to water source points (reservoirs) within the corresponding WDN [169,170]. Additionally, the multi-agent concept can evaluate the homogeneity of an already partitioned network using alternative clustering approaches [171].

In a recent study, Serafeim et al. [172–175] developed an advanced tool for water distribution networks (WDNs) partitioning into district metered areas (DMAs) or pressure management areas (PMAs) [176]. They employed the hierarchical clustering methodology introduced by Deidda et al. [177], grounded on Ward's method [178–181]. Additionally, the tool incorporated topological proximity constraints, such as nodes' altitude, to curtail excessive partitioning of the original water network. The introduced approach boasts several strengths. Firstly, it uses the original pipeline grid as a connectivity matrix, thus minimizing the risk of unrealistic clustering outcomes. Secondly, it upholds statistical rigor and impartiality by solely relying on statistical metrics, without depending on user-defined weighting factors. Lastly, it is easy to implement, demanding minimal processing power, rendering it well-suited for engineering applications.

5. Discussion and Conclusions

The estimation of the lost volume of water in water distribution networks (WDNs) employs three primary approaches: (a) the top-down (or water balance) approach, (b) the burst and background estimates (BABE) approach, and (c) the bottom-up (or minimum night flow, MNF) approach. The water balance approach, introduced by the International Water Association (IWA), estimates water losses by subtracting the legitimate (i.e., authorized) consumption from the system input volume (SIV), while distinguishing between real losses (RL) and apparent losses (AL) using semi-empirical assumptions. The BABE approach, which was also proposed by the IWA, categorizes leaks into bursts and background losses, estimating their respective volumes based on average flow rates and leakage duration. The bottom-up approach (or MNF estimation method) decomposes the estimated MNFs into real losses (RL) and users' night consumption (UNC), using statistical metrics to provide reliable leakage estimates during the nighttime of the low consumption periods of the year. The probabilistic minimum night flow method is recommended as the most rigorous approach when high-resolution flow timeseries are available, as it does not rely on semi-empirical or empirical assumptions. In addition, it allows for confidence interval estimation of flow minima, which are representative of leakage rates, making it suitable for engineering applications.

The causes of leaks along the pipelines of water distribution networks can be attributed to a variety of structural, external, internal (hydraulic), and maintenance-related factors. Structural considerations encompass a wide range of parameters such as pipeline diameter, wall thickness, material composition, and age, as well as the density of pipe connections. Larger diameters and thicker walls generally offer greater resistance to mechanical stresses, and several studies indicate an inverse correlation between pipeline failures and diameter, with material strength and age also playing pivotal roles. Additionally, soil subsidence, erosion, and fluctuating groundwater levels can compromise pipeline integrity, while temperature fluctuations and external loads from traffic or other natural causes can stress pipelines, leading to cracks and failures.

Internal (or hydraulic) factors, including flow velocity and pipe pressure, significantly affect leakage rates. For example, high water velocities may damage pipes as well as their joints, while pressure differences between the inner and outer sides of the pipe wall intensify leakage rates due to the developed pressure gradient.

Effective leakage management in water distribution networks is essential to uphold their financial viability and operational reliability, considering the impacts associated with the lost volume of water. The most common leakage mitigation and performance optimization approaches have focused on pipeline repair and replacement strategies, while advancements in hydraulic engineering have led to the development of innovative techniques to enhance leakage reduction, such as water distribution networks' partitioning into DMAs and/or PMAs.

Initiatives involving pipe repair and replacement constitute critical decision-making processes when addressing structural or functional degradation of pipelines. Repairing strategies (management option 1) target localized breaks, while pipe replacement interventions (management option 2) consider various factors such as timing, prioritization, and economic viability. Combining elements of the aforementioned two types of pipeline management strategies, based on of the pipeline network's characteristics, available funds, and associated risk tolerance levels, may result in an effective plan, striking a balance between long-term infrastructure resilience and economic considerations.

On the other hand, water networks partitioning into DMAs and/or PMAs aims at reducing the operating pressures so as to minimize leakages. Advanced methodologies, including heuristic and spectral graph clustering, offer promising avenues for optimizing network partitioning and enhancing leakage management. Recently introduced approaches incorporate advanced tools for segmenting water distribution networks by applying rigorous statistical methods enhanced with topological proximity constrains for realistic and efficient network partitioning into DMAs and/or PMAs.

In summary, although novel engineering solutions and advanced methodologies can significantly minimize the volume of leakages in water distribution networks, robust legislative measures must be implemented so as to properly address this prevalent issue. A comprehensive legal framework can ensure the enforcement and implementation of the appropriate maintenance procedures and the efficient allocation of funds for repairs and replacements, with respect to industry standards, so as to prevent future leaks. Hence, a collective endeavor involving policymakers, stakeholders, and industrial leaders is imperative to enact and enforce regulations that address the fundamental causes of leakages and ensure the long-term stability and resilience of water infrastructure.

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