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An Expeditious Campaign of Field Experiments for Preliminary Analysis of the Hydraulic Behavior of Intermittent Water Distribution Networks

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Abstract: The paper describes the results of a field experimental campaign carried out on the intermittent water distribution system (WDS) of a small municipality in southern Italy. In a novel way, as compared to the existing literature, the monitoring campaign covered the whole cycle of operation of the WDSs. In total, 8 days of experiments were carried out between June and August of the year 2019. Simultaneous measurements of water level and outflow from the municipal reservoirs, and nodal pressures were collected in order to analyze the water distribution network (WDN) behavior during the intermittent supply. The collected data give us a proper understanding of the functioning of the WDS during the whole cycle of intermittent supply, also providing the base for future proper network modelling under intermittent operation. In addition, preliminary analysis of inequity in water distribution among users and water leakages throughout the network are derived from the collected data. Finally, limitations of the study as well as potential for future research developments are discussed.

Keywords: intermittent water distribution; monitoring campaign; inequity in water distribution

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1. Introduction

Water Distribution Systems (WDSs) are complex infrastructures designed to process, store and supply potable water to consumers continuously and reliably.

However, hundreds of millions of people around the world receive water intermittently [1–4]. Under intermittent conditions, WDSs can supply water for time periods less than 7 days a week and/or less than 24 h a day. In both cases, the WDS behaves as an intermittent system, with network end users that have access to water in a discontinuous way. This can be a harbinger of inconvenience in the distribution of water to the population.

Water resource scarcity is considered as the primary motivation that determines intermittent operation; however, the reduced availability of water and, more in general, all the environmental constraints constitute only one aspect of a multi-dimensional problem [5–9]. Other causes of intermittent supply may be caused by power shortages, pollution accidents, ineffective operation of the system and leakage due to deteriorated infrastructure.

From a hydraulic point of view, intermittent WDSs undergo processes during which the water distribution network (WDN) is cyclically subject to filling and emptying phases with pipes switching alternatively from free-surface to pressurized flow conditions. In general, intermittent water supply implies (*i*) an initial phase of network filling at the beginning of the water supply period, (*ii*) a phase of functioning under pressure conditions and, (*iii*) a phase of network emptying when the supply period is over. The duration of each of these phases depends on both the water supply modalities and the network characteristics.

During the filling process, the network pipelines are progressively filled by water. The filling process can evolve in different ways based on the characteristics of the network and on the supply from the source; pressure and free-surface flow conditions may co-exist in the network because of differences in pressure due to head losses [10].

The modality of the network filling has a deep impact on the spatial/temporal water distribution in the system and determines which users may access to water first, and which have to wait for longer time. Normally, users try to collect water as much as possible during the supply hours; in most cases, they have private tanks and store water to be used as backup source during no-supply hours. The filling of private tanks is usually controlled by float valves, so that the inflow to the tanks does not stop unless they are full. This modality of water supply determines the withdrawal of large water volumes from the network in relatively short times, thus causing an increase in head losses with a consequent increase in pressure deficits in the system. The massive presence of private tanks (which represents an additional storage capacity), also increases the time to complete the network filling. When the supply period is over, the network emptying phase completes the cycle of intermittency of the WDS.

Besides all the technical impacts and consumers' behavior described above, there is also a high social cost of intermittent WDSs resulting from inequity in water distribution among the network users [11–13]. The non-uniformity in space and time in water allocation among users is directly associated with the non-uniformity in pressure distribution among the network nodes. This problem is even exacerbated in large networks characterized by high gradients in elevation. The topological structure of the network may determine some locations to be more advantaged than others because of their elevation or their proximity to the supply reservoir [14,15]. As pointed out by [16], each node is characterized by a different water reception time (i.e., the time needed for the node to receive the water from the beginning of the supply period), depending on the node position in the network. For the most disadvantaged nodes, the reception time may be greater than the supply duration, thus determining that, in principle, such nodes will receive less water than others. Moreover, the inequity in water distribution of intermittent WDSs [14,16] may be sharpened by the progressive installation of private tanks by the end users [17–21].

The analysis of the scientific literature shows that, in comparison to continuous WDSs, intermittent WDSs are less studied. Notably, the available literature on the topic mostly focuses on the analysis of causes and consequences of intermittent WDSs at a macro-scale [2,5]. Other studies concern the hydraulic behavior of such systems [10,21,22] including pressure transients during the filling phase [23].

Conversely, experimental data on intermittent WDSs are very scarce, and the unavailability of experimental databases represents an important gap in this area of research. This gap prevents the correct validation of software tools for the modelling of intermittent WDSs, as well as the subsequent simulation of design/managing strategies aimed at mitigating the negative impacts of intermittency in water supply. Additionally, the lack of data does not allow the use of methods for leakage estimation developed for continuous WDSs. Therefore, the availability of field data from real-scale intermittent WDSs is of paramount importance to cross this gap and to improve knowledge on their hydraulic behavior.

In order to contribute to explore the complex processes occurring during the operation of intermittent WDSs, in the summer season of the year 2019, an expeditious field experimental campaign was planned and conducted on an intermittent WDS in southern Italy. The experimental campaign consisted in collecting water levels, flow and pressure measurements in the intermittent WDS for the whole cycle of operation of the network.

This paper presents the main results of the experimental campaign and discusses the impacts of the intermittent distribution on the hydraulic behavior of the water distribution network and its users. To the best of the authors' knowledge, this is the first monitoring campaign on intermittent WDSs that covers the whole cycle of operation including the phases of filling, operation and emptying of the network. The availability of the data obtained from the experimental monitoring enabled the operator of the water distribution system to understand problems due to intermittent operation to try improving water distribution strategies. The detail of the information presented in the paper may support the calibration of models of the WDN for the correct simulation of the different phases of the intermittent water distribution.

The paper is articulated as follows. The methodological section includes a detailed description of the case study and the modalities of the campaign of experiments. Then, the results of the experimental monitoring are presented and discussed, including limitations of the methods used and results obtained. Finally, conclusions are drawn with potential for future research developments.

2. Materials and Methods

2.1. Case Study Description

The campaign of measurements was carried out on the intermittent water distribution system of Mirabella Imbaccari, a municipality located in the outback of Sicily, southern Italy. The objective of the experimental campaign was to support the municipality in the analysis of the whole cycle of operation of the intermittent WDS to identify potential for improvements in the supply operational strategy of the network.

The town of Mirabella Imbaccari is located at elevations ranging between 440 m and 530 m a.s.l. The WDS supplies water to a resident population of about 5000 inhabitants. Remarkably, population almost doubles during the months of July and August because of the return from abroad to the homeland of migrant workers for summer holidays.

Water sources of the WDS include both springs and wells providing mean flow (in the year) of about 17 L/s in total. Notably, the three main springs (Monastra 1, Monastra 2 and Dragofosso) globally supply about 8 L/s to the WDS while the municipal wells (Aranzulla, Mirci and Cutrona) supply the remaining 9 L/s (see Figure 1). The three wells are located at elevations of 485, 480 and 468 m a.s.l., respectively.

Figure 1. Municipality of Mirabella Imbaccari—location of springs, wells and reservoirs.

The WDN (see layout in Figure 2) is supplied by two municipal reservoirs (reported as R1 and R2 in Figures 1 and 2), both located in the zone of major elevation in the southern part of the town. Figure 2 also shows the paths of the supply mains (dashed lines) adducting flows from wells and springs to the two reservoirs.

Figure 2. Layout of the WDN of the municipality with indication of sources and reservoirs, and site of installation of pressure gauges and flow meters.

Both reservoirs are structured in multiple tanks. The oldest reservoir (R1) is made of two twin tanks characterized by rectangular base (5.90 m width × 15.50 m length), and total height of 4.30 m. Each tank has a maximum storage capacity of about 350 m^3. The bottom of the two tanks is at the field campaign elevation (528 m a.s.l.). There are two outlet pipes (size DN200) both equipped with a gate valve that connect the reservoir R1 to the WDN. Reservoir R2 is of more recent construction and is placed adjacent to R1.

The flow that comes from the sources supplies reservoir R1 in a continuous way during the 24 h of the day. A pipe that works as overflow device of R1 and positioned at 3.72 m from the bottom of the tanks of R1 connects R1 to R2 allowing excess water in R1 to overflow to R2 (Figure 2). Therefore, R2 operates in series with R1 and is filled only when the reservoir R1 is full. Additionally, R2 is connected to the WDN by means of two outlet pipes (DN200) equipped with valves. Actually, these outlet pipes convey the outflow from R2 in the two outlet pipes of R1, thus supplying the WDN.

The principal WDN in the town is represented by of an old system of mains. During the 1990s, additional portions of the network were constructed and connected to the old network in order to supply water also to new built areas of the town. Such more recent portions of the WDN mainly constitute the secondary and capillary water distribution networks.

Information concerning the network characteristics (nodes elevation, length and material of the pipes) as well as the detail of the distribution of the population in the area served by the WDN were obtained from data provided by the municipality and from the results of the field surveys. The WDN has a substantial closed-loop layout with open branches that supply flows to more peripheral parts of the urbanized area. The network is characterized by high gradients in elevation with differences between the highest (reservoir R1 in the south of the town) and the lowest points served (at about 423 m a.s.l.). From the altimetry viewpoint, the WDN can be roughly divided into two portions that supply zones at different elevation: the “high zone” of the old town located in the south-east part of the municipality, and the “low zone” including areas of more recent urbanization in the north-west part of the municipality. The two zones are connected by means of a main (DN200) just downstream of the two reservoirs (see Figure 2). A servo-controlled gate valve is installed in the connection main that facilitates conveying the whole flow from the reservoirs to the high zone only, thus disconnecting the low zone of the WDN according to the operational scheduling of the intermittent supply. Primary feeder pipes of the network (including large pipes with diameters between 50 and 200 mm) are in spheroidal cast iron while the secondary feeders (corresponding to smaller diameters and capillary network) are in high-density polyethylene (HDPE). Because of the age of the system and the reduced maintenance, the WDN is characterized by relatively high values of water leakage. An estimation by the municipality based on historical data indicates average water leakage in the order of 30–40% of the flow supplied.

There are no significant hydro-demanding activities in the town, as the economy of the area is mainly based on small-scale artisanal and commercial activities.

From the operational point of view, the WDN provides water to the end-users intermittently. Intermittency in water supply is both in time and space. The supply period varies slightly during the days of the week and during the months of the year. Generally, water is provided to users according to a complex prefixed schedule: from Monday to Friday (ferial supply modality, FSM), the gate valves at the outlet pipes of R1 are opened early in the morning (around h 6:00 a.m.). After about 3.5 h from the beginning of the supply period (approximately when the water level inside R1 drops down to about 1.5 m), gate valves at the outlet pipes of R1 are closed, while those of R2 are opened. During this phase of the supply period, water flows to the network only from reservoir R2, while R1 is progressively refilled by the inflow from the sources. In the afternoon (between h 12:00 and h 4:00 p.m.), the two valves of the outlet pipes of R2 are closed almost completely until the new full opening of the day after. Actually, the two valves are left a little bit open to allow a minimum flow to be conveyed to the WDN during evening and night hours.

Generally, in the first 2.5 h (out of 3.5 h) of the supply period the servo-controlled gate valve in the pipe that connects the networks of the high and low zones is maintained closed to assure suitable pressure levels in the old town. Then, at around h 8:30 a.m., the valve is switched on, and part of the flow is also conveyed to the portion of the network of the low town.

On Saturday and Sunday (weekend supply modality, WSM) the distribution schedule is slightly different. Indeed, at h 6:00 a.m. the reservoirs supply the whole network (both zones) while the schedule remains the same of the ferial days for the rest of the day.

Because of the intermittency in the water supply, almost all households of the WDS are equipped with private tanks. Such tanks store water during supply hours and work as back-up sources during non-supply periods. The results of a recent survey and successive estimations made by the water service of the municipality have shown that the private tanks in the network globally provide a distributed storage capacity close to 1000 m^3 in total. The major part of private tanks are standard 1 m^3 tanks (per household) and most of them are located on the rooftop of the buildings. Instead, a minor part of them are located in the basement of the houses. Typically, private tanks are equipped with a floating valve that allows inflow from the WDN to be controlled. Specifically, the inflow to the tank is maximum when the water level in the tank is below the level of the floating valve; conversely, with the exception of eventual local malfunctioning, the inflow is expected to be zero when the level of the floating valves is achieved. The presence of such large number of household tanks impacts severely the distribution of the water, sometimes making it impossible to operate the WDN under appropriate conditions of pressure.

2.2. The Campaign of Experiments

The campaign of field measurements in the WDS was conducted during the summer season of the year 2019. The analysis of the hydraulic behavior of the system required the preliminary acquisition of the geometric, topological and hydraulic characteristics of the elements of the network, e.g., pipes, reservoirs, private tanks, etc.

The campaign of monitoring involved the acquisition of low-cost measurements with the aim of identifying the WDN behavior during the whole cycle of intermittent supply. Experimental measurements concerned the filling phase, the phase of operation under pressure conditions, and the phase of network emptying.

The monitoring included measurement of the water levels in the reservoirs, flow discharges and pressures in specific pipes and nodes of the WDN. Water levels in the two reservoirs were observed with time intervals of about one hour, through direct reading of hydrometric stages. Accuracy of water level measurements was ±0.01 m. Moreover, electromagnetic flow meters (model RPmag, range 50–1100 m^3/h, accuracy ±0.5%) (f1 and f2 in Figure 2) were installed at the outlet of the two reservoirs for the instantaneous measurement of the flow supplied to the WDN. The two flowmeters provided measures of the total flow supplied to the network by the two reservoirs, R1 and R2. Since the water supply schedule provided the alternated operation of the two reservoirs, values of the outflow from R1 and R2 were determined, separately. Pressure gauges (model Pedrollo MR10, range 0–10 bar scale, accuracy ±0.5%) were mounted in different points of the network. In total, 7 gauges were installed; 3 of them (gauges 1, 2 and 3) were placed in the high zone of the network; the remaining four gauges 4, 5, 6 and 7 were installed in the low zone of the network.

The sites of installation of the pressure gauges allowed us to cover the whole urban area and the whole range of elevation and pressure in the WDN. For ease of installation and monitoring, the pressure gauges were mounted at the points of withdrawal of the households, normally at the base of the buildings. Figure 2 shows the location of the points selected for pressure measurements, while elevations of the same points and the respective zone of the WDN are reported in the following Table 1.

Table 1. Elevation of the installation sites of the pressure gauges.

Pressure Gauge	Elevation (m a.s.l.)	Zone of the WDN
1	510	high
2	500	high
3	493	high
4	483	low
5	472	low
6	463	low
7	439	low

Globally, 8 days of experiments were carried out from June to August 2019 (2 days in June, 3 days in July, and 3 days in August). Different days of the week were considered with the aim of catching eventual intra-week variations on the consumer habits and on the operation of the WDS.

Due to the floating population during the summer season, the number of end-users served by the WDS grew during the period of time covered by the measurement campaign, with a peak of population occurring in the month of August.

Table 2 specifies the supply modalities (FSM or WSM) during the experiments. It is worth noticing that for the experiment of day n. 7 (Sunday, 11 August) the network was supplied according to the FSM modality (high zone of the WDN first) instead of that typical of weekend days (WSM). Based on information provided by the municipality, this change in the operational modality was due to the remarkable increase in the water demand in the days of mid-August.

Table 2. Summary of the experimental conditions for the 8 days of experiments.

Experiment n.	Month	Day	Supply Modality	Monitoring Time
1	June	Saturday 15	WSM	6:00 a.m.–6:00 p.m.
2	June	Sunday 16	WSM	6:45 a.m.–6:45 p.m.
3	July	Tuesday 2	FSM	6:00 a.m.–6:00 p.m.
4	July	Wednesday 3	FSM	6:00 a.m.–3:00 a.m.
5	July	Friday 5	FSM	6:00 a.m.–3:00 a.m.
6	August	Friday 9	FSM	6:00 a.m.–3:00 a.m.
7	August	Sunday 11	FSM	6:00 a.m.–5:00 a.m.
8	August	Wednesday 14	FSM	6:00 a.m.–3:00 a.m.

For any day of experiments, the outflow from the reservoirs was continuously measured by the flow meters, and data were collected and stored in a data-logger using a time step of 1 min. The mean daily inflow (from water sources) to reservoir R1 was evaluated based on a set of volumetric measurements in the reservoir. The water level in the tanks of the two reservoirs was measured with a time step of about 1 h. Nodal pressures at the 7 measurement sites in the WDN were collected on every hour approximately; modalities and times of opening and closure of gate valves in the WDN were also recorded.

3. Results and Discussion

In this section, the behavior of the network is analyzed in detail for the two different modalities of intermittent operation of the WDN (FSM and WSM).

The results of the experiment of Sunday, 16 June (see Table 2) are shown in detail as an example of the behavior of the WDN during the weekend days. On that day, the whole network (both the high and the low zones) was supplied from the beginning of the supply period at h 6:00 a.m. Conversely, the results of the experiment of Wednesday, 3 July, are selected to show the operation of the network during ferial days. As already introduced, during that day, the first part of the supply period concerned only the high zone of the network. For the two days of measurement considered, the data collected (outflow discharges from the reservoirs, water levels in the reservoirs and nodal pressures at the 7 gauges) are summarized and discussed, as follows.

As explained in Section 2.1, the whole WDN is supplied through the two outlet pipes of R1 that collect outflows from both reservoirs. Because of the network layout, the flow is not equally distributed between the two outlet pipes. Specifically, data from the field survey revealed that the outlet pipe in which flow meter f2 is installed supplies only a relatively small loop of the network with very small population served, while most of the WDN is supplied by the outlet pipe in which flow meter f1 is installed. The long-term analysis of the collected data confirmed that this last outlet pipe normally conveys most of the flow from the two reservoirs to the WDN (mean 30 L/s) while the other pipe (flow meter f2) conveys a mean flow of about 0.4–0.5 L/s (almost constant for all the days). For this reason, only the results of the flow monitoring of f1 are reported in the following of the paper.

Figure 3 and Figure 5 show the graphs of the outflow discharge Q (L/s) from the reservoirs recorded by flow meter f1 (Figure 3a and Figure 5a) and the graphs of water level h (m) observed in the two reservoirs (Figure 3b and Figure 5b) on 16 June and 3 July, respectively. Corresponding observed nodal pressures p (m) in the WDN are also reported in Figure 4 and Figure 6 for the two days. Gauged nodes located in the high and in the low zones of the network were grouped together in Figure 4a,b and Figure 6a,b, respectively.

The figures show the behavior of the WDN during the different phases of the intermittent supply. The analysis of the time series recorded by f1 (Figure 3a) for Sunday, 16 June, provides clear identification of how the network operated on that day. The gate valve at the reservoirs are opened approximately at h 6:00 a.m. and closed at h 11:50 a.m. Therefore, the opening period covered about 1/4 of the day (about 6 h out of 24).

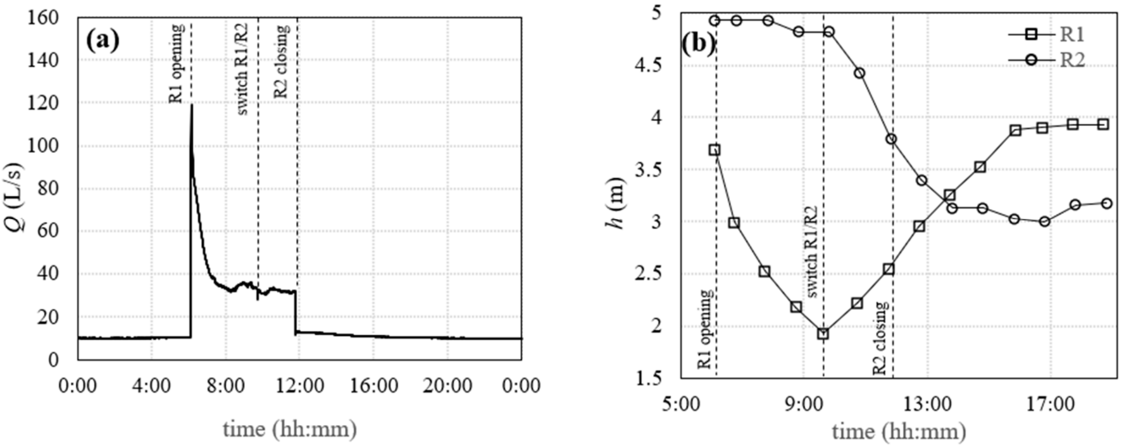

Figure 3. (**a**) Outflow discharge Q from the reservoirs, and (**b**) water level in R1 and R2. Experiment of Sunday, 16 June 2019.

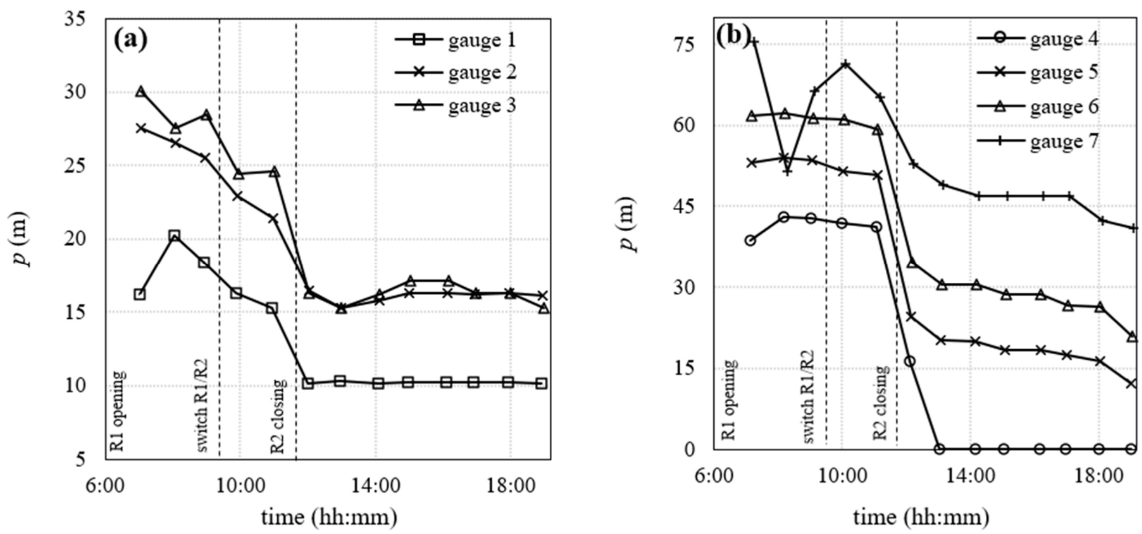

Figure 4. Nodal pressures p at nodes located in the (**a**) high, and (**b**) low zone of the WDN during the experiment of Sunday, 16 June 2019.

The servo-controlled valve between the high and the low zone of the WDN was open from the beginning of the supply, thus allowing the system to supply the whole network. Figure 3a also shows that the value of the flow recorded by the flowmeter was not zero during the rest of the day, as one would expect. Indeed, both before and after the opening period, the gate valves in the outlet pipes of R2 were left partially open in order to provide (base) flow to the network also during evening and night hours. In this way, the operators of the water distribution system allow the end users of the WDN to re-fill their private tanks during the night. In addition, this base flow avoided the complete emptying of the network, thus reducing problems associated with the entrance of air in the pipes. Values of the outflow from R2 during the periods h 0:00–h 6:00 a.m. and h 11:50 a.m.–h 12:00 p.m. ranged from 9.5 L/s up to 13 L/s with a mean value of about 10.5 L/s.

Figure 3a shows the outflow discharge to increase abruptly from the base value of the night up to about 120 L/s as the gate valves were opened at h 6:00 in the morning. The peak of discharge reveals the high water demand associated with the process of filling of the network (which was partially empty at that hour of the morning). The figure also shows that, after the peak, the flow decreased rapidly down to about 35 L/s in less than two hours (at h 8:00 a.m.). The decrease in the discharge is in agreement with the advancing of the process of filling of the network including private tanks installed by the households in the town. Remarkably, the rate of the decrease in the discharge (slope of the curve) reduces significantly starting from h 8:00 a.m., due to the reduction in water demand because of the progressive closure of floating valves of private tanks. Interestingly, the curve of the discharge maintains almost the same trend of decrease for several hours after the gate closure at the reservoir (at h 11:50 a.m.), up to the complete filling the private tanks in the night. At about h 11:00 p.m., the value of the discharge achieves its minimum (close to 9.5 L/s) and remains almost constant for the rest of the night hours. At that time of the day, most of the private tanks are full, thus revealing that the remaining flow is likely to represent water leakages. Simple considerations based on the system water balance supports such evaluation.

The effect of the opening of the valve is also shown in Figure 3b in terms of water levels in the two municipal reservoirs. The water level inside R1 at about h 6:00 a.m. was equal to about 3.70 m. From this time on, the figure shows that the water level decreases, and the reservoir is progressively emptied. Measurements of water level in reservoir R1 in the time window h 6:00–h 9:40 a.m. follow a line with negative slope (Figure 3b) during the process of network filling. In the same time window, the water level in reservoir R2 shows only a slight decrease due to the condition of partial opening of the corresponding gate valves at the outlet of the reservoir. The switch of operation from reservoir R1 to reservoir R2 can be observed at about h 9:40 a.m. (Figure 3b). At that time the water level inside R1 was equal to 1.93 m, and the operators of the WDN closed the gate valves of reservoir R1 and opened those of R2. Figure 3a shows a slight discontinuity in the curve of the outflow from the reservoirs that is related to the closure/opening of the gate valves. The switch of operation is more evidently shown in Figure 3b. Starting from h 9:40, the water level inside R1 returned to increase (linearly) due to the relatively constant inflow to the reservoir from the source. Conversely, the water level in R2 decreased due to the outflow from the reservoir.

Finally, at about h 11:50 a.m., the gate valves of R2 were partially closed and the supply period was over. Figure 3a shows that, at that hour, the flow drops down to the base value and the slope of the curve of the water level in R2 decreases (Figure 3b). Remarkably, at around 4:00 p.m., R1 was full and started to overflow to R2. Figure 3b shows that water level in R1 remained almost the same, while the level in R2 started to increase again. The filling process of R2 continued until the beginning of the supply period of the successive cycle of distribution. The curve of the outflow during supply hours (between h 6:00 to h 11:50 a.m.) reported in Figure 3a points out another interesting aspect of the intermittent operation of the water distribution system. The flow supplied to the WDN does not follow the pattern typical of continuous demand-driven systems. In fact, the outflow from the reservoirs is not directly influenced by users' demand because of the interposition of private tanks installed at the different households.

The graphs in Figure 4 show peculiarities of pressure trends observed in the network nodes. Two distinct periods of the day can be identified for all the pressure gauges: the first period (during supply) is characterized by relatively high pressure values, while the second period is characterized by low values of pressure (starting from around h 12:00 p.m.) for all the monitored nodes. Gauged nodes from 1 to 3 (Figure 4a) placed in the high zone of the network show maximum pressure values to range between 15 and 30 m. Conversely, gauges from 4 to 7 (Figure 4b) in the low zone of the WDN show values of the pressure that may achieve up to 75 m for the lowest node (gauge 7). Maximum pressure values were recorded after the opening of gate valves of reservoir R1. During the supply period (h 6:00–h 9:40 a.m.) pressure progressively decreased in all the nodes because of the high water demand during the filling process. As expected, at the end of the supply period, the figures show pressure to quickly drop down to minimum values that remain almost constant during the rest of the day. It is worth noticing that, due to gate valves of reservoir R2 being partially open even during evening and night hours, nodal pressure values did not drop to zero. Node 4 only (the highest in the low zone of the network) showed values of pressure close to zero during the non-supply period (Figure 4b). The field survey revealed that this behavior can be related to the high demand in this nodal area of the WDN because of a large number of oversized tanks and to a high water leakage levels.

Concerning the experiments of 3 July 2019, Figure 5a shows that gate valves of reservoir R1 were opened at h 6:00 a.m. In a different way from the previous case, the supply period started with the connection between high and low zone of the network being closed. Therefore, the high zone of the network was the only part of the WDN served by the reservoir until h 8:45 a.m. The figure points out the discharge from R1 to increase gradually from about 5 L/s to about 10 L/s. Coherently, Figure 5b shows that the water level in R1 decreased from 3.9 m (value at 6:00 a.m.) to about 3.65 m (value at 08:45 a.m.). The emptying of R1 during this part of the day was slow because the water demand was limited to a relatively small part of the network with users' tanks already full by the night flow. At the same time, the level in R2 increased because of the overflow from R1. At h 8:45 a.m., the servo-controlled valve was opened, and also, the low zone of the network was supplied; the graph of the outflow discharge shows the occurrence of a peak close to 140 L/s. As expected (Figure 5b), the slope of the curve of R1 changed with water level that dropped down to about 1.80 m in less than 2 h (from h 8:45 a.m. to h 10:35 a.m.). At h 10:35, the gate valves of R1 were closed, and those of R2 were opened. From this moment on, the water level in R2 decreased, while reservoir R1 was progressively re-filled by the inflow from the sources. Finally, at about h 12:45, the gate valves of reservoir R2 were partially closed; the connection valve between the high and the low zone of the network was simultaneously closed. The end of the supply period is identified in Figure 5a in correspondence to the time at which the outflow discharge drops down to about 7 L/s. This determined a decrease in the slope of the curve of the water level in R2 reported in Figure 5b. Close to h 5:00 p.m. the water level in R1 achieved the level of the overflow; from that time on, the water level of R1 did not change significantly, while the water level in R2 started to increase until the beginning of the supply period on the successive day.

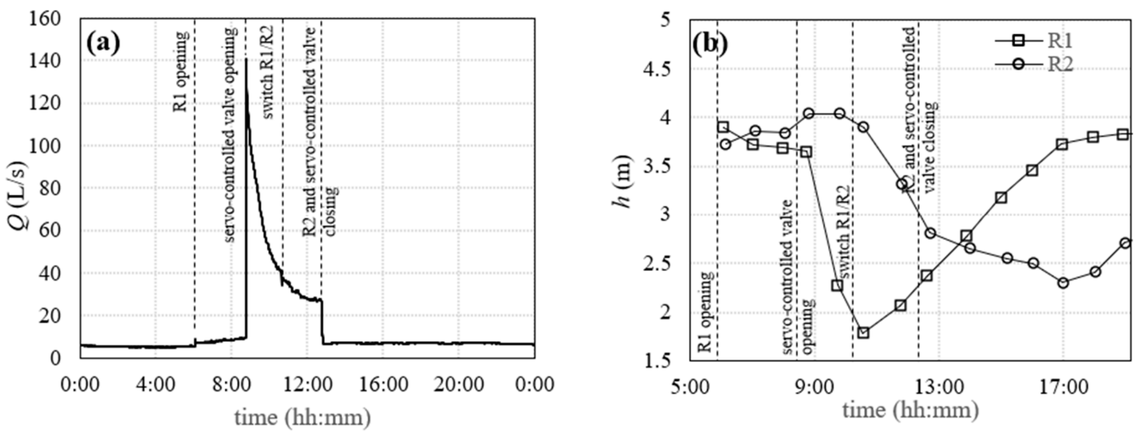

Figure 5. (**a**) Outflow discharge Q from the reservoirs, and (**b**) water levels in R1 and R2. Experiment of Wednesday, 3 July 2019.

As nodal pressure is concerned, different behavior of gauged nodes in the high and in the low zone of the network can be identified. Nodes of the high zone of the network (Figure 6a) show a decreasing trend of the nodal pressure during the supply period (h 6:00–h 12:45 a.m.), similarly to that observed for the 16 June (Figure 4a). After the supply period, nodal pressure remained almost constant. However, both the maximum pressure values (at the beginning of the supply period) and the asymptotic values (after the end of the supply period) were greater than the values showed in Figure 4a for all the gauged nodes in the high zone of the network. This behavior is related to the partitioning of the WDN in the FSM (i.e., the servo-controlled valve between high and low zone of the network was closed during the no-supply period). Nodal pressures in the low zone of the network (Figure 6b) show a different trend in comparison to the same nodes on day 16 June (Figure 4b). Except for node 7, the other gauges show zero pressure values before and after the supply period. Indeed, the low zone of the network did not receive the base flow from reservoir R2 due to the closure of the servo-controlled valve. Therefore, the low part of the network remained totally empty during the no-supply hours. After the opening of the servo-controlled gate, pressures increased with a linear trend until h 10 a.m. (Figure 6b) and remained almost constant until the end of the supply period (h 12:45 a.m.). Later, after the closure of the servo-controlled valve, pressure values dropped down again to zero because of the emptying of the network. Gauged node 7 shows a different behavior during the no-supply hours as compared to other nodes in the same zone of the network. In particular, both before and after the supply period, pressure values for node 7 were almost constant and above 30 m. Since gauged node 7 is the lowest one in the low zone (Table 1), this behavior could be related to the presence of a depressed part of the network that allows for local storage of water in the condition of low network pressure.

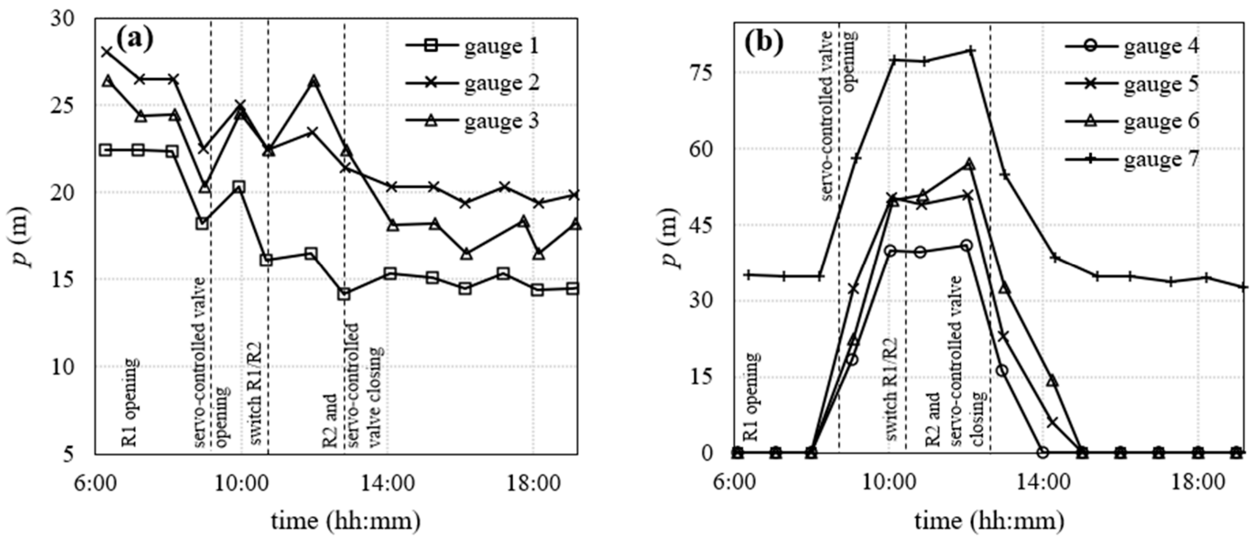

Figure 6. Nodal pressures *p* at nodes located in the (**a**) high and (**b**) low zone of the WDN during the experiment of Wednesday, 3 July 2019.

Effect of the opening of the servo-controlled valve at h 8:45 can also be observed in Figure 6a. Indeed, at that time, nodal pressures in the high zone of the network show a sudden decrease that can be ascribed to the start of the supply of the low zone of the network. Similar considerations of those set out above can be made for the other experimental days reported in Table 2.

Globally, the knowledge of processes occurring in the analyzed intermittent system during the whole cycle of operation would enable the operator of the WDN to set up proper strategies for improved water distribution. As an example, the results reported in Figures 4 and 6 allow us to discuss aspects concerning the spatial-temporal distribution of the flow in the intermittent WDN. Despite the supply period on day 16 June starting at h 6:00 a.m., some nodes had to wait until h 8:00 a.m. to reach the maximum pressure. Indeed, both node 1 in the high zone of the network (Figure 4a) and node 4 in the low one (Figure 4b) show an increasing trend of the pressure from h 7:00 to h 8:00 a.m. It is worth noticing that first pressure measurements were registered at h 7:00 a.m. Therefore, the transitory phase during the network filling process was not picked up by the monitoring process. Most likely, nodal pressure between h 6:00 and h 7:00 a.m. were also lower than values registered at h 7:00 a.m. The non-uniformity in water resource distribution in space and time is even exacerbated on day 3 July because of the FSM. Figure 6b shows that, except for node 7, all nodes in the low zone of the network did not receive flow until h 9:00 a.m. (i.e., 3 h after the beginning of the supply from R1). Moreover, since the servo-controlled valve between the high and the low zone of the network was closed at about h 12:45, nodal pressures in the low zone dropped to zero between h 14:00–h 15:00 because of the emptying process of this part of the network. This means that the real supply window for users in the low zone of the network had a duration of 5–6 h. This shows that the non-uniformity in pressure distribution throughout the network constitutes a major cause of inequity in water distribution among the users of the studied WDN. Therefore, the operator of the water distribution network can set up structural measures and interventions to reduce pressure non-uniformity within the WDN. An opportunity in this sense could be the introduction of pressure Real Time Control (RTC) techniques in the WDN [24].

Another aspect deserves further discussion. A first preliminary consideration about the amount and the distribution of water leakages throughout the network can be derived from the analysis of the outflow from the reservoirs. At the end of the supply period, the outflow rapidly drops down to the value of the base flow. At the beginning of the no-supply period, this base flow could allow the refilling of users' private tanks. Indeed, Figure 3a shows a decreasing trend of the outflow curve from h 12:00 (end of supply) to h 20:00. This behavior is less evident in Figure 5a since the base flow serves a smaller portion of the network (high zone only). However, starting from h 08:00 p.m. and continuing until the beginning of the supply period of the successive day, the base flow remains almost constant. Since during night hours water, demand due to real users' consumption in the town can be neglected, outflow from the reservoirs can be ascribed to network water leakages. Indeed, the high peak flow registered at the beginning of the supply period (Figures 3a and 4a) would mean that, despite the base flow, the water network is almost empty in the morning, thus supporting the hypothesis of water leakages. Remarkably, minimum night flows (h 4:00 a.m.) on 16 June and 3 July were about 10 L/s and 5 L/s, respectively. Therefore, about 50% of the total water leakage can be ascribed to the high zone of the network, which is served by 26% of the total network pipe length. This behavior is in line with information provided by the municipality that indicates most of the water leakage to be concentrated in the high zone of the network (the oldest). Further evaluations including modelling analysis through the adoption of a specific model of the intermittent WDN may enable verification of such preliminary estimations concerning the amount of water leakage in the network. Given the current status of the infrastructure, a more rational managing of the WDN may provide for the total closure of gate valves from the reservoirs during night hours. Indeed, the base flow could be limited to the first few hours after the end of the supply period, allowing the refilling of private tanks. In this way, the total volume of water leakage may be reduced with an increase in the level of service of the water supply.

Although results are site specific, methodology and lessons learned by the experiments can be transferred to the analysis of such processes in other intermittent systems.

It should be remarked that the analysis was limited by the available financial budget allocated for the field campaign. The adopted setup for the experiments can be considered as appropriate for expeditious campaigns aimed at preliminary analysis of the behavior of the network. Conversely, more sophisticated monitoring setup would be needed for further steps aimed at improved analysis of pressure distribution in space and time in the intermittent WDN to better catch pressure fluctuations occurring during the transient phases of network filling and emptying.

4. Conclusions

The results of expeditious field experiment to explore the hydraulic behavior of the intermittent water distribution network of a small municipality in the south of Italy were presented in this paper. In a novel way, as compared to the body of the existing literature, the experiments covered the whole cycle of operation of the WDSs. In total, 8 days of experiments were carried out between June and August of the year 2019. The case-study WDN is supplied intermittently for about 6 h per day, both during week and weekend days. Most of the households of the WDN are equipped with private tanks as back-up sources of water during non-supply periods.

The analysis of simultaneous measurements of water levels and outflows at the municipal reservoirs, as well as pressure measurements at specific nodes of the WDN allowed us to investigate the behavior of the network during the intermittent supply.

The following main conclusions can be drawn:

- Intermittent operation of the WDN implies cyclical phases of filling and emptying of reservoirs, pipes and private tanks with co-existing pressure and free surface conditions in different portions of the network. Such phases determine pressure fluctuations throughout the network that may generate non-uniform distribution of water flows among users (thus inequity) and stress on the infrastructure; the possibility of a correct interpretation and modelling of the functioning of intermittent WDSs cannot disregard the knowledge of the whole cycle of intermittent water distribution;
- The network layout and the presence of users' private tanks exacerbate the problem of inequity in the water distribution. The experiments have shown users in the WDN to be supplied in a non-uniform way with some of them that may wait up to 3 h from the beginning of the supply period to obtain access to water;
- The availability of experimental data for intermittent WDNs is of paramount importance to understand the complex processes of such systems in order to setup proper operational strategies to mitigate the negative impacts of supply intermittency; although results are site-specific, methodology and lessons learned by the experiments can be transferred to the analysis of such processes in other intermittent systems;
- Research efforts are also required to implement a hydraulic model of the analyzed case-study network. Appropriate calibration of the model by means of the available experimental data and successive simulations would improve our understanding of the network behavior, proper estimation of water leakages, as well as identification of optimal operational strategies to improve water distribution in the WDN.

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