Leakage Management for Water Distribution Infrastructure: Results of DMA Experiments in Ottawa, ON

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Leakage Management for Water Distribution Infrastructure – Report 2: Results of DMA Experiments in Ottawa, ON

Osama Hunaidi

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National Research Council Canada
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Canada

EXECUTIVE SUMMARY

Leakage in municipal water pipe networks is a significant problem that has economic, environmental, legal, social and political consequences. Most municipalities need to manage leakage in their pipe networks. The need to do so became more urgent in recent years due to water shortages caused by recent draughts; increasing demand; environmental, social and political pressures; escalating energy cost and looming regulatory requirements.

Leakage management generally involves water audits, leak detection or monitoring, pressure control, and leak location and repair. In extreme cases, it may involve pipe rehabilitation or replacement. Water audit procedures are well established, e.g., the IWA standard audit which was implemented for Regina in this study using a GIS-based software system. Pressure reduction is rarely used for leakage management in Canada. This will likely not change, at least in the near future, because of concerns about meeting fire fighting requirements; potential revenue loss; high maintenance of pressure control equipment and long-term effectiveness of pressure control. Municipalities are able to locate most leaks successfully using acoustic equipment, e.g., electronic listening devices and correlators that advanced dramatically in recent years.

Most municipalities in Canada that have active leak detection policies utilize periodic acoustic surveys to detect leakage in pipe networks; very few municipalities use district-metered areas (or DMAs) to detect or monitor leakage. In recent years, encouraged by the widespread use and success of DMAs in the United Kingdom, an increasing number of municipalities are considering DMAs. However, it’s not known if present DMA practice is directly applicable to municipal water pipe networks in Canada.

Several important components currently used in DMA flow analysis were based on data collected primarily from water pipe networks in the U.K. and Germany and their applicability to Canadian networks was uncertain. Pipes in Canada are larger in diameter and therefore are expected to have different background losses,

leak frequencies and flow rates. In addition, patterns of night water demand in Canada are believed to be different from European patterns due to differences in population life style and residential plumbing.

The objective of this study was to develop enhanced leakage management methods for municipal water pipe infrastructure in Canada, with emphases on the district metered-area method. The study involved extensive fieldwork in Ottawa and Regina under controlled conditions to determine residential night demand, background and recoverable leakage rates, and effect of pipe pressure on leakage; and to evaluate analytical procedure(s) for component identification of DMA night flow. Planned fieldwork in Halifax could not be performed due to administrative difficulties at NRC.

Experimental DMAs were set up in two areas in Ottawa that were representative of the city's dominant pipe types. These included: (i) an ~1850 service connections ductile iron pipe DMA in the Orleans area in the eastern part of the city, and (ii) an ~900 service connections cast iron pipe DMA in the high pressure Meadowlands area in the western part of the city. Experimental fieldwork was carried out in 2006 and 2007. Following are the main findings and conclusions.

The best-fit power relationship between background leakage and pressure for ductile iron pipes was $L_{Background} = 0.0203 \times P^{1.11}$, where leakage is in L/connection/h and pressure is in psi. The N_1 exponent of this power relationship is significantly lower than the exponent of 1.5 used in current practice. For cast iron pipes, the best-fit power relationship between background leakage and pressure was $L_{Background} = 0.0075 \times P^{1.351}$.

There was significant variation in background leakage rate from night to night and sometimes over the same night that did not correspond to pressure stepping in the older Meadowlands area. Probably, this could be attributed to variable plumbing losses, especially toilet leakage in old homes. The best-fit power relationship between the highest suspected toilet leakage was $L_{Toilet} = 0.0103 \times P^{1.43}$. Possibly, this formula underestimates toilet leakage since measurements were made over few nights only and subsequently the highest possible flow rate may have been missed.

Background leakage rates predicted by current practice with estimates at a reference pressure of 71 psi (50 metres) and N_1 exponents obtained in this study were reasonably close to rates based on the best-fit power relationship of measured rates. A better overall agreement, taking into account flow measurements in both Regina and Ottawa, was obtained by slightly adjusting constants in the equation used in current practice as follows:

$$\text{Night background leakage} = \left(A \times L_m + B \times N_c + C \div 15 \times L_p \times N_c\right) \times \left(\frac{P}{71}\right)^{N_1}, \text{ in L/h}$$

where P is average pipe pressure in psi; N_1 is equal to 0.55, 1.11, 1.35 and 1.5 for asbestos cement, ductile iron, cast iron and PVC pipes respectively; L_m is total length of distribution pipes in km; N_c is total number of service connections; L_p is average length in metres of service connection pipes between curb stops and customer water meters; and A, B and C are constants equal to 24 L/km/h, 1. 5 L/connection/h and 0.4 L/connection/h corresponding to rates of leakage components at 71 psi (50 metres) pressure in distribution mains, service connection pipes from mains to curbstops and 15-metre long service connection pipes after curbstops, respectively. N_1 for PVC pipes was not measured in this study but was based on current practice.

Difficulties were encountered in proving the boundaries of test areas used to measure background leakage rates and N_1 exponents in the older Meadowlands area. It was likely that some boundary valves were passing water but this could not be detected using acoustic listening equipment while the valves were subjected to a moderate pressure differential (zero-pressure tests were not permitted). It may be that the flow rate of passing water was too low to create detectable noise or that the gates of some valves were worn out and moved somehow after the valves were sounded. The latter could happen due to the removal of valve keys that rested on top of valve nuts during sounding or due to the change of differential pressure across valve gates after sounding, during pressure stepping.

Residential night demand was generally higher than the estimate of 1.7 L/household/h used in current practice. This was especially the case during the lawn-watering season, with demand being up to 5 times the currently used estimate.

Overall average residential night demand over the period 1:30-3:30 AM outside the lawn-watering season in the older Meadowlands test area was higher by about 1 L/household/h than demand in the newer Orleans area. Subsequently, it may be necessary to use DMA-specific residential night demand in order to accurately determine the rate of recoverable leakage based on analysis of DMA night flow. In the absence of DMA-specific data, it is proposed that analysis of night flow be based on DMA supply flow rate averaged over the period 1:30-3:30 AM or 2:00-3:00 AM (instead of using minimum 60-minute moving average) and an average residential demand of 3 and 2 L/household/h for older and newer areas, respectively.

There can be significant spurious recoverable leakage determined from the analysis of night flows during the lawn-watering season. Also, there can be significant night-to-night variation in the rate of recoverable leakage. In one instance, recoverable leakage rate increased with decreasing pipe pressure. The significant spurious leakage and its variation are a result of the night use of lawn water sprinklers. Results obtained in this season should be treated with caution. Outside the lawn-watering season, there can also be spurious recoverable leakage, especially in areas with older homes, which are more likely to have leaking toilets.

Detailed procedures were presented for setting up either temporary or permanent DMAs to estimate recoverable leakage rate, with or without automatic meter reading (AMR) systems.

Poor results were obtained for night flow components calculated analytically using a system of linear algebraic equations formulated using power exponents used in current practice or measured in this study and assuming constant residential night demand.

Fieldwork in Ottawa to compare different leak detection strategies could not be performed. Instead, description of different acoustic leak detection strategies, discussion of their pros and cons, and reported experiences with their performance were presented. In addition, interesting results emerged from acoustic listening and correlation surveys that were undertaken to determine the source of high leakage detected in the cast iron and asbestos-cement pipe DMAs in Ottawa and Regina, respectively.

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LIST OF SYMBOLS

$^\circ C$	Degrees Celsius
$\varnothing$	Diameter
A	Constant equal to 20 L/km/h, corresponding to rate of background leakage at 71 psi pressure in distribution mains
AC	Asbestos Cement
AMR	Automatic Metre Reading
AWWA	American Water Works Association
B	Constant equal to 1.25 L/connection/h, corresponding to rate of background leakage at 71 psi pressure in service connections to from mains to curbstops
c	Marginal cost of lost water ($/m^3)
C	Constant equal 0.5 L/connection/h, corresponding to rate of background leakage at 71 psi pressure in 15-metre long service connection pipes between curbstops and customer water metres
CARL	Current Annual Real Losses
C_{survey}^{DMA}	Cost in dollars of acoustically surveying a DMA
CI	Cast Iron
DI	Ductile Iron
DMA	District Metered Area
FFT	Fast Fourier Transform
F_o	Frequency of unreported leaks per km of pipe per year
GIS	Geographic Information System
h	Hour
Hz	Hertz, unit of frequency defined as the number of cycles per second
ILI	Infrastructure Leakage Index
IWA	International Water Association
km	Kilometre
L	Litre
$L_{Background}$	Background leakage
L_{DMA}	Total length of distribution mains in km in a DMA in km
L_m	Total length of distribution mains in km
L_p	Average length of service connection pipes in metres between curbstops and customer water metres

LIST OF SYMBOLS (CONT'D)

L_s	Total length of service connection pipes in metres between curbstops and customer water metres
m	Metre
N_1	Exponent for power relationship between leakage and pressure
N_c	Total number of service connections
P	Pressure
PRV	Pressure Reducing Valve
psi	Pressure unit, pounds per square inch
PVC	Polyvinyl Chloride
R	Average volume of water lost in m^3 per year per leak
sub-DMA	A small area of a DMA
$T_I^{optimum}$	Theoretically optimum intervention period for acoustic leak surveys
UARL	Unavoidable Annual Real Losses
UKWIR	United Kingdom Water Industry Research

1 INTRODUCTION

Municipal water pipe networks deteriorate naturally with time and subsequently lose their initial water tightness. Deterioration is caused by corrosive environment, soil movement, poor construction practices and workmanship, fluctuation of water pressure, and excessive traffic loads and vibration. Water is lost due to leakage in different components of the networks: transmission pipes, distribution pipes, service connection pipes, joints, valves, fire hydrants, and storage tanks and reservoirs. In addition to physical losses due to leakage, many pipe networks suffer from so called apparent losses. These are caused by customer meter under registration, accounting errors, and unauthorized water use.

Water loss in municipal water pipe networks is a common problem in Canada. On average, about 20% of the water produced by municipalities in Canada is lost, mostly due to leakage in the pipe network (Environment Canada, 2004). Losses in the United States are reported to range from 15% to 25%, of which about 60% to 75% is recoverable leakage (Vickers, 1999).

Management of leakage in water pipe networks has several potential benefits. In addition to helping municipalities meet water demand, lowering of leakage helps to reduce water quality breaches that may result from the entry of contaminants via leaks. Also, reducing leakage helps to decrease the high cost of energy wasted on treatment and pumping of lost water (Colombo and Karney, 2002). The energy-wasting aspect of leakage is important as significant savings can be realized. Energy to supply water is the second largest cost after labour for water systems in developed countries, and the cost may easily consume 50% of a municipality's budget in the developing world (James et al., 2002).

Leakage in municipal water pipe networks has economic, environmental, legal, social and political consequences. Therefore, most municipalities need to manage leakage in their pipe networks. The need to do so became more urgent in recent years due to several factors including:

- Diminishing water resources and subsequent shortages caused by more frequent and prolonged droughts due to climate change. This problem is particularly serious in the Canadian Prairies, e.g., the 1999-2004 episode. The Prairies are likely to face a severe drought within the next couple of decades (CBC, 2006).
- Increasing demand for water due to population growth.
- Escalating energy cost.
- Increasing awareness by customers of the environmental and social effects of water loss, e.g., effect on energy usage, contamination, and property damage and disruptions due to emergency repairs.
- Emergence of best practice guidelines and new technologies for effective management of leakage.
- The significant and sustained reduction in the rates of lost water realized by utilities, e.g., those in the U.K., which implemented best practices and new technologies to manage leakage.
- High water losses are being increasingly looked upon as an indicator of inadequate maintenance activity and ineffective utility management.
- Looming regulatory requirements to manage leakage.

Leakage management involves water audits, leak detection or monitoring, pressure control, and leak location and repair. In extreme cases, it may involve pipe rehabilitation or replacement. Water audit procedures are well established, e.g., the IWA standard audit which was implemented for Regina in this study using a GIS-based software system. Pressure reduction is rarely used in Canada for leakage management and this will likely not change, at least in the near future. Municipalities are able to locate most leaks successfully using acoustic equipment, e.g., electronic listening devices and correlators that advanced dramatically in recent years.

Most municipalities in Canada that have active leak detection policies rely on periodic acoustic surveys to detect leakage; very few municipalities use district-metered areas (or DMAs) to detect or monitor leakage. In recent years, encouraged by the widespread use and success of DMAs in the United Kingdom, an increasing number of municipalities are considering DMAs. However, it was not certain if current DMA practice is directly applicable to water pipe networks in Canada.

Several important components currently used in DMA flow analysis were based on data collected primarily from water pipe networks in the U.K. and Germany and their applicablity to Canadian networks was not certain. Pipes in Canada are larger in diameter and therefore are expected to have different background losses, leak frequencies and flow rates. In addition, patterns of night water demand in Canada are thought to be different from European patterns due to differences in population life style and residential plumbing. Accurate information about night residential demand, background leakage, and pressure effect is needed in order to reliably determine the rate of recoverable leakage in a DMA.

The following points can be further shortcomings of international leakage methods if applied to North American systems without proper modification: (i) IWA's model for estimating Unavoidable Annual Real Losses (UARL) does not account for soil type, pipe burial depth and climate – these have significant effects for systems in Canada and Northern United States, (ii) the 500 litres per hour threshold for technically undetectable leaks is dated – significant advances in acoustic leak detection equipment were made in recent years which dramatically lowered the threshold, and (iii) the UARL model does not account for different leak survey procedures, e.g., acoustic mapping or correlation-based surveys can detect a larger number of leaks than simple listening surveys. Inaccurate UARL adversely impacts leakage management since it may lead, for example, to underestimates of recoverable leakage and unrepresentative infrastructure leakage indices.

2 OBJECTIVE

The objective of this project was to develop enhanced leakage management methods for municipal water pipe networks in Canada. Emphases were on the district metered-area method and included:

(i) Determination of night residential demand.

(ii) Development of an empirical model for estimation of background leakage rates and/or development of analytical procedure(s) that can derive it directly, e.g., analytical modelling of night flow rates measured under different pipe pressures.

(iii) Derivation of relationships between leakage rate and pipe pressure.

(iv) Comparison of the performance of different acoustic leak detection methods.

3 SCOPE

The project involved extensive measurement and analysis of flow and pressure nightlines for several residential district metered areas at two municipal water pipe networks in Canada having different but typical regional characteristics. The selected networks were those of Ottawa and Regina. Planned fieldwork in Halifax could not be performed due to administrative difficulties at NRC. Two district-metered areas (DMAs) were created in each water pipe network. Experimental fieldwork was performed under controlled conditions to: (i) measure night residential demand and if possible evaluate indirect statistical procedure(s) for calculation of residential demand, (ii) measure background and recoverable leakage rates, (iii) establish leakage-pressure relationships, and (iv) evaluate analytical procedure(s) for component identification of night flow. Planned fieldwork in Ottawa to compare different acoustic leak detection strategies could not be performed because the city as a result of stretched resources and unexpected heavy workload could not provide the required support. Instead, description of different acoustic leak detection strategies, discussion of their pros and cons, and reported experiences with their performance are presented in Appendix A.

In this report, findings and conclusions based on fieldwork carried out in Ottawa are presented and discussed. Details of fieldwork; instrumentation and software; measurement and analysis procedures are also provided. First, however, an overview of leakage management is presented. Findings and conclusions based on fieldwork in Regina are presented in a companion report.

4 OVERVIEW OF LEAKAGE MANAGEMENT

Management of leakage consists of the following four main components: (i) quantifying total water loss; (ii) detection or monitoring of leakage; (iii) locating and repairing leaks; and (iv) pipe pressure management. In extreme situations, leakage management may involve pipe rehabilitation or replacement but this is not considered here.

The total amount of lost water is quantified by conducting a system-wide water audit, known internationally as water balance. Procedures for conducting water audits were published by the American Water Works Association (AWWA, 2009) and by the International Water Association (Alegre et al., 2000) and (Lambert, 2003).

Like financial audits that account for all the debits and credits of a business, water audits account for all water flowing into and out of a utility's water delivery system. An audit can be performed over an arbitrary period, but normally is computed annually over 12 months. Audits provide a valuable overall picture of the various components of consumption and loss, which is necessary for assessing a utility's efficiency regarding water delivery, finances, and maintenance operations. In addition, water audits are necessary for planning other leakage management components.

In Regina, the IWA standard water balance was implemented in this study in a GIS-based software system, presented in Appendix B. This system integrates data from the water network asset inventory, water supply supervisory control and data acquisition (SCADA) records, automated meter reading system (AMR), and other water use data to automatically generate water balance reports, quantify water loss and recoverable leakage, and calculate key water loss performance indicators, including the infrastructure leakage index (ILI).

Leak detection can be achieved using district-metered areas. This involves dividing the pipe network into well-defined areas and monitoring water flow supplied to each area. The boundaries of DMAs can sometimes occur naturally but generally have to be created by closing appropriate valves. The size of a typical DMA can be between 500 and 3000 service connections (Hunaidi & Brothers, 2007a). Night flow rates of DMAs are monitored on quarterly, monthly or continuous basis if data loggers connected to DMA flow meters are equipped with remote communication devices (e.g., cellular modems). Guidelines for setting up, maintaining, and monitoring the leakage of DMAs were published by UKWIR (1999) and IWA (2007).

Leakage in a DMA is suspected if the minimum night flow rate exceeds a certain threshold. The latter is determined as the sum of the flow rate of water used by all nighttime commercial and industrial users in the district, flow rate of water used by all residences based on an estimate of average night flow rate per household, and background leakage rate. DMAs make it possible to quickly and efficiently identify areas of pipe networks that suffer from excessive leakage, which are then targeted for acoustic leakage detection and localization operations. Analysis of night water flow can also be used to refine (or check) the accuracy of water audits.

District metered areas are not commonly used in Canada. Most municipalities that have active leak detection policies rely on periodic acoustic surveys to detect leakage. In these surveys, the water pipe network is checked for leaks from end to end either by listening to or by correlating acoustic noise induced by leaks at various contact points with pipes. All areas of the pipe network are surveyed whether or not leakage is suspected. Municipalities that have active leak detection policies usually survey their pipe networks at a frequency of about two years.

An alternative to periodic acoustic listening or correlation surveys, especially in noisy parts of cities, is the use of acoustic noise loggers. These are compact units composed of a vibration sensor (or hydrophone) and a programmable data logger. Noise loggers are deployed in groups of six or more at adjacent pipe fittings (e.g., fire hydrants and valves 200 to 500 m apart) and left overnight. The units are normally programmed to collect pipe noise data between 2:00 and 4:00 AM. They are collected the following day and recorded data is downloaded to a personal computer before the loggers are deployed at the following location. The data is analyzed statistically, e.g., frequency analysis of leak noise levels, to detect the presence of leaks. Recent models of acoustic noise loggers can be deployed permanently. Leak noise is measured nightly and processed using on-board electronics, the result is stored in memory for later transmission wirelessly to a roaming vehicle or a permanent receiver.

The economic viability and leak detection effectiveness of temporarily or permanently deployed noise loggers is questionable. Van der Klejj and Stephenson (2002) found that both permanently and temporarily deployed loggers are not an economical alternative to skilled and well-equipped leak inspectors. For network-wide coverage, permanent loggers had a minimum payback period of 25 years. When the loggers were used in temporary mode (i.e., moved from one survey area to the next), they were three times less efficient than acoustic surveys. Van der Klejj and Stephenson also reported that the number of leaks found by noise loggers and by general listening surveys were similar; however, the loggers failed to detect approximately 40% of leaks found by detailed listening surveys.

The exact position of a leak is commonly pinpointed using ground microphones and leak noise correlators and sometimes by using non-acoustic methods, such as thermography, ground-penetrating radar, and tracer gas (Hunaidi et al., 2000). Pinpointing leaks can be time consuming. Therefore, in the case of DMAs, leak areas are narrowed down to a few pipe sections before pinpointing them. A procedure known as “step testing” can be used to achieve this. Step testing involves the monitoring of the district meter’s flow rate while successively closing valves within the DMA, starting with the valve farthest away from the meter. A significant reduction in the flow rate is an indication of leakage in the last shut-off section. Step testing has to be performed at night and can be time consuming and dangerous. Also, closing/opening of valves may lead to water quality problems and sudden pressure fluctuation. In recent years, its use has dwindled in favour of acoustic surveys using noise loggers, acoustic listening tools, or leak noise correlators.

Pipe pressure affects leakage in a number of ways and a substantial reduction in leakage can be realized by pressure reduction (Report 26, 1980) and (Thornton, 2003). Theoretically, the flow rate of a fluid through an opening is proportional to the square root of the pressure differential across the opening, provided the dimensions of the opening remain fixed. However, the effective area of the opening may enlarge with pressure. Therefore, much greater reductions in leakage may be realized than predicted by the square root relationship, especially for small leaks from joints and fittings in most pipe types and large leaks in plastic pipes (Lambert, 2001). A linear relationship between pipe pressure and leakage rate is widely used by leakage management practitioners.

Pressure reduction is rarely used in Canada for leakage management and this will likely not change, at least in the near future. Municipalities are usually concerned about meeting fire fighting requirements; potential revenue loss; high maintenance of pressure control equipment and long-term effectiveness of pressure control.

5 DESCRIPTION OF TESTS

Test Sites

Measurements of night flow and pressure were performed in summer and fall 2006 and spring 2007 at two specially created DMAs in Ottawa, each having a different pipe type. The first DMA was in the Orleans area in the eastern part of the city (see Figure 1 for aerial view). The DMA has 21.74 km of distribution pipes constructed in the late 1960s, 1970s and 1980s, of which 84.8% is ductile iron (DI), 7% is polyvinyl chloride (PVC), and 8.2% is undefined pipes. The DMA has 1834 service connections, the majority of which are residential except for 2 schools and a large retirement home (see APPENDIX C for further information). The DMA includes a large residential complex comprising 240 apartments. The number of boundary valves that were needed to completely isolate this DMA was 5 (Figure 2). The size of this DMA is close to that of typical DMAs, which normally have 2000 service connections.

Flow and pressure were also measured in a small sub-area (sub-DMA) of ductile iron pipe DMA. The small area consisted of ~2.33 km of distribution pipes (almost all is ductile iron) and 298 service connections, all of which are residential except for two schools. The number of boundary valves that were needed to completely isolate this DMA was 6 (Figure 3). Total pipe length was suspected to be short; actual length was re-read from DMA plan and was ~3.2 km (used in this report.)

The second DMA was in the Meadowlands area in the western part of the city (Figure 4). This is a high-pressure area and is known to have pipe breakage and leakage problems. The DMA has 15.31 km of distribution pipes constructed in the 1960s, 70s and 80s, of which 66.6% is cast iron (CI), 15.6% is ductile iron (DI), 10.2% is polyvinyl chloride (PVC), and 7.2% is undefined pipes. The DMA also has 909 service connections, most of which are residential except for 4 schools and 25 small commercial outlets (see APPENDIX C for further information). The DMA includes 2 large residential buildings comprised of 447 apartments. The number of boundary valves that were needed to completely isolate this DMA was 2 in addition to 20 permanently closed valves for the pressure zone (Figure 5).

Flow and pressure were also measured in an initial sub-area of the cast iron pipe DMA (tests conducted in November 2006) that consisted of ~2 km of distribution pipes and 220 service connections, all of which is residential. The number of boundary valves that were needed to completely isolate this DMA was 17 (Figure 6). In subsequent tests in spring 2007, flow and pressure were also measured in a sub-

DMA (purple zone) that consisted of 3.6 km of mostly cast iron pipes and 181 service connections, all of which are residential except for two small commercial outlets and a school (no apartment buildings). This sub-DMA was later enlarged (purple and blue zones) to an area comprised of 5.75 km of distribution pipes of which 73.1% is CI, 7.7% is DI and 19.2 is PVC. Total number of services in the enlarged sub-DMA was 392 all of which is residential except for two small commercial outlets and a school (no apartment buildings).

The above DMAs are representative of pipe types and conditions commonly found in Ottawa. The city's water pipe network is comprised of about 2391 km of distribution pipes, of which 39% are cast iron, 34% are ductile iron, and 26% are PVC. The network has 178,704 service connections and it services 765,000 people. Average pipe pressure in the network is 70 psi (47.6 m). Average volume of water pumped into the network is 368 ML/day and the marginal cost of water is ¢4.6/m^3. The infrastructure is assumed to be in good condition and current leakage management strategy is passive.

Setup of DMAs

District metered areas were isolated at night by temporarily closing all boundary valves of the area between approximately 11:00 PM and 5:00 AM. Water was supplied to isolated areas via an above ground bypass by running a short 2-inch fire hose (~10 m) connected to taps on either side of a boundary valve inside a manhole (Figure 7). Above ground, fire hoses were connected to a portable rig that included a flow meter, pressure reducing valve (PRV) and a pressure gauge (Figure 8). Pipe pressure was recorded at the DMA inlet as well as at a fire hydrant near a point where pressure was equal to the average value of pressure in the whole DMA. Pressure was corrected by adding 4 psi to account for a pipe depth of 2.75 m below the pressure sensor attached to the hydrant.

The integrity of boundary valves, i.e., their water tightness, was checked nightly prior to flow measurements. To do this, pressure inside DMAs was reduced below that in surrounding areas by 20 to 30 psi. A valve that's not tightly seated creates a hissing sound under differential pressure. The integrity of valves was checked by listening for flow noise using an acoustic listening device on a solid key that rested on the valve (Figure 9). Initially, few valves were found to be passing in each DMA. Tight seating of most passing valves was restored by either exercising them several times before closing them tightly. If this did not help, in some cases simply turning valves backward a couple of turns restored proper seating. If none of this helped, the boundary of the DMA was adjusted to exclude passing valves. Following the initial check, only valves that were reopened between night measurements were re-checked.

The register of the inlet flow meter was read manually at the beginning and end of the period over which flow information was recorded. Totals based on flows acquired by data loggers and manual readings were compared as an accuracy check. Differences were always negligible.

Instrumentation and Software

All instrumentation and software used for measuring, recording and analyzing flow and pressure were off-the-shelf and commercially available. Water flow into DMAs was measured using one or more of the following flow meters:

- 3/4-inch Neptune T-10 positive displacement flow meter having a pulse factor of 17.07 pulses per litre; low flow rate of 1 litre per minute at 95% accuracy; and normal operating range between 2.8 and 114 litres per minute at 100% accuracy (±1.5%)
- 2-inch Neptune T-10 positive displacement flow meter having a pulse factor of 1.98 pulses per litre; low flow rate of 3.83 litres per minute at 95% accuracy; and normal operating range between 9.5 and 606 litres per minute at 100% accuracy (±1.5%)
- 3-inch Neptune Trident turbine flow meter having a pulse factor of 0.0352 pulses per litre; normal operating range between 19 and 1703 litres per

minute at 100% accuracy (±1.5%); and maximum intermittent flow rate of 2120 litres per second

Calibration of flow meters was checked at the city's meter shop by city staff prior to field measurements.

Magnetic drive signals of flow meters were digitized and recorded using Neptune FloSearch II transmitter inserted between the flow meter and its register and MeterMaster data logger model 100. For comparison, magnetic drive signals were also digitized and recorded using MeterMaster Model 50 strap-on magnetic sensor and Radcom model LoLogLL Vista data logger. The MeterMaster logger operates in pulse-count mode only while the Radcom logger can operate in both pulse-count and pulse-interval-timing (PIT) mode. PIT overcomes accuracy problems suffered by simple pulse counting for measuring low flow with meters having insufficient pulse output. Data files were exported from loggers to Microsoft Excel spreadsheets for analysis and display.

A Singer model 106-PR pressure reducing valve (1 ½ inch in 2006 and a 1 inch in 2007) with a low flow stabilizer model 26 was used to control pipe pressures in DMAs.

Test and Analysis Procedures

Residential night water demand and background leakage rates were established based on collective measurement of water use by a group of 200 to 400 residences. Initially, it was planned to perform flow measurements for the sub-DMAs while residential curb-stops were open and then closed. However, closing curb stops was unfeasible because of operational constraints.

Flow measurements for the ductile iron pipe sub-DMA in Orleans were performed in summer and fall 2006 on several nights between approximately 11:30 PM and 5:00 AM. Initial measurements were undertaken in the cast iron pipe sub-DMA in Meadowlands in fall 2006 and further measurements were undertaken in spring 2007. Prior to conducting these measurements, leak detection surveys were carried out and all detected leaks were repaired. Only one fire hydrant leak was detected in the Orleans sub-DMA and repaired prior to flow measurement.

Because of the small number of residences in the Orleans and Meadowlands sub-DMAs, it's very likely that there would be several short intervals with no water being consumed by residences (except for plumbing losses). Therefore, background leakage rate was considered to correspond to the minimum measured flow rate. Average residential night water demand was estimated as the average hourly flow rate (calculated over a 1- or 2-hour period) minus the minimum flow rate divided by the number of residences in the sub-DMA. Average residential night demand based on these measurements excludes losses from residential plumbing.

Water loss due to leakage in DMAs was evaluated as average flow rate of water supplied to DMA minus average flow rate due residential demand based. Both minimum moving 60-minute average supply flow rate and average flow over a fixed period were used.

Residential night water demand could be estimated indirectly by statistical analysis of 1-week long (or more) high-resolution measurements of DMA flow nightlines. The principle behind this method is that unless there are significant fluctuations in pressure, water demand due to leakage in DMAs remains almost constant at night. Therefore, fluctuations in the flow nightline of a DMA will be fully attributable to demand from residences in the district (assuming commercial and industrial use is insignificant or can be accounted for entirely). In this statistical method (Creasey et al., 1996), residential demand is assumed to be dominated by a known short fixed-volume event, e.g., toilette flush, and that the average total demand is constant.

The measurement period is divided into equal short intervals 1 to 5 minutes in length and measurement period should be long enough to have at least 250 intervals (preferably 500). Total flow volume measured in each interval is converted into an equivalent number by dividing it by the average volume of a toilette flush (or whatever the dominant fixed volume event is). The number of fixed-volume events in each interval is assumed to follow a Poisson distribution. The mean of this type of distribution is equal to its variance.

Since night flow fluctuation is attributed to residential demand only, the mean number of events corresponding to the latter is taken equal to the variance of the total flow expressed in number of events per interval. Residential night demand is equal to the mean number of events multiplied by the average event volume. Leakage level is estimated by subtracting mean residential demand from mean total demand. This analysis was not possible in Ottawa because resources were directed to higher priority measurements. Measurements required for the statistical analysis are excessive and staff time allocated to the project was limited.

Background and recoverable leakage rates and residential night demand could be determined analytically from DMA night flow measurements under significantly different pipe pressures. Background and recoverable leakage components respond differently to variation in pressure and assuming that residential night demand is not dependent on pipe pressure (e.g., due to fixed volume toilette flushes), a model could be established to separate these components (APPENDIX D).

The relationship between leakage rate and pipe pressure were derived from DMA night-time flow measurements during at least 3 different pipe pressures (e.g., 80, 60, 40, psi), before and after leak detection and repair.

Water flow and pipe pressure were recorded at 5-second long intervals between approximately 11:30 PM and 5:00 AM. Recorded flow information was used to determine minimum, average and maximum flow rates for stationary 1-mintute long intervals. A 60-minute moving average was also determined based on 1-minute averages. Recorded pressure information was averaged over stationary 1-minute long intervals. All recorded flow and pressure nightlines are presented in Appendix E.

6 RESULTS AND OBSERVATIONS

Background Leakage

Ductile iron pipes

As can be seen from the flow nightline obtained from a preliminary flow measurement in the ductile iron pipe sub-DMA in Orleans on 7 June 2006 (Figure 10), there were short periods over which the 1-minute minimum flow remained almost constant. This was considered to indicate that there was no residential demand during these periods and subsequently the minimum flow rate was indicative of background leakage rate. Background leakage rates of ~6 L/connection/h were obtained at pipe pressures of ~53 and 59 psi, and ~8 L/connection/h at ~77 psi. These rates are about 3 times the rate estimated using current practice for a system in good condition.

Background leakage rate is estimated in current practice using the following equation based on background leakage at 71 psi (50 metres) reference pipe pressure (Lambert et al., 1999) and a power leakage-pressure relationship with exponent equal to 1.5 (Lambert, 2001):

Eq. (1): $$\text{Night background leakage} = \left(A \times L_m + B \times N_c + C \div 15 \times L_p \times N_c\right) \times \left(\frac{P}{71}\right)^{N_1}$$, in L/h,

or

Eq. (2): $$\text{Background leakage level} = \left(\frac{A}{c_d} + B + C \div 15 \times L_s\right) \times \left(\frac{P}{71}\right)^{N_1}$$, in L/connection/h

where P is average pipe pressure in psi; N_1 is 1.5, L_m is total length of distribution mains in km; N_c is total number of service connections; c_d is connection density in number of connections per km of distribution main; L_p is average length of service connection pipes in metres between curbstops and customer water metres; L_s is total length of service connection pipes in metres between curbstops and customer water metres and A, B and C are constants corresponding to rates of background leakage components at 71 psi pressure in distribution mains, service connections to from mains to curbstops and 15-metre long service connection pipes between curbstops and customer water metres equal to 20 L/km/h, 1.25 L/connection/h and 0.5 L/connection/h, respectively.

If diurnal pressure fluctuation is insignificant, i.e. head losses are small which is the case for a distribution system with large pipes, the daily background leakage rate is equal to:

Eq. (3): $$\text{Background leakage} = 24 \times \left(A \times L_m + B \times N_c + C \div 15 \times L_p \times N_c\right) \times \left(\frac{P}{71}\right)^{N_1}$$, in L/day

For systems in average and poor conditions, background leakage rates estimated using current practice are two and three times that estimated for a system in good condition, respectively.

The high rate of background leakage measured during the night of 7 June 2006 could be due to a running lawn-watering sprinkler in the sub-DMA. A single sprinkler may consume ~30 litre per minute at 77 psi pressure, which could spuriously raise background leakage by ~6 L/connection/h. Based on this, actual background leakage may be equal to 2 L/connection/h, which is close to the value predicted by Eq. (2) for a system in good condition. However, this is mere speculation.

Significant flow oscillation was observed in the flow nightline obtained from preliminary flow measurements in the ductile iron pipe sub-DMA in Orleans during the night of 7 June 2006 (Figure 10). It was suspected that this was due to PRV hunting under low flows. To investigate this, flow in the ductile iron pipe sub-DMA in Orleans was re-measured on 26 June 2006 but with the PRV bypassed. During this flow measurement, it was observed that the telltale of the flow meter sometimes rotated backwards (for up to 5 seconds). This indicates backflow from the sub-DMA. Most likely, backflow was caused by sudden high demand in adjacent areas of the distribution pipe network. Therefore, minimum flow rates might not correspond to actual background leakage rates.

As can be seen from Figure 11, there were several minima below 1 L/connection/h but they did not last for more than 5 seconds. Therefore, for the purpose of estimating background leakage rate, minimum flow rates that did not last for a period of at least 30 seconds were not considered. Close inspection of Figure 11, indicates that there were three such long periods at 1:57, 2:01 and 2:45 AM. There was slight fluctuation in minima over these periods but the average value was consistently equal to ~3.2 L/connection/h, compared to the rate of 2.9 L/connection/h obtained from Eq. (2) used in current practice for estimating background leakage for distribution pipe networks in good condition.

To prevent backflow from the ductile iron pipe sub-DMA in Orleans, flow was re-measured on 27 June 2006 while passing through a fully open PRV to introduce some head loss. The telltale of the flow meter at the inlet of the sub-DMA never turned backwards; however, on several occasions it almost came to a complete stop. Clearly, the PRV acted as a backflow preventer. Also, as can be seen from the flow nightline in Figure 12, there were two or more long enough periods over which minimum flow rate, corresponding to background leakage, relatively remained constant at 2.9 L/connection/h. Average pressure in the sub-DMA during these measurements was ~92.5 psi.

Flow in the ductile iron pipe sub-DMA in Orleans was also re-measured on 28 June 2006 while passing through a check valve and bypassing the PRV. As can be seen from Figure 13, the flow came to a complete stop on several occasions for no more than 5 seconds. However, it can also be seen that there were two or more long enough periods over which minimum flow rate corresponding to background leakage relatively remained constant at ~2.9 L/connection/h. Average pressure in the sub-DMA during these measurements was ~92.5 psi. It’s interesting to observe that at 3:30 AM the flow increased suddenly by about 5 L/connection/h. This probably was to draw by a lawn water sprinkler.

Flow measurements in the Orleans ductile iron pipe sub-DMA to establish the relationship between background leakage rate and pressure were also performed in late October 2006, outside the lawn-watering season. On 17 October, flow was measured under normal operating pressure of ~92 psi, bypassing the PRV. As during measurements in June 2006, it was observed that the telltale of the flow meter sometimes almost stopped or rotated backwards, indicating backflow from the sub-DMA. However, as can be seen from the flow nightline (Figure 14), there were two or more long enough periods over which minimum flow corresponding to background leakage almost remained constant at ~3.04 L/connection/h, e.g., between 3:04 and 3:08 AM. This leakage rate is close to the background leakage rates measured in June 2006 at similar pipe pressure.

Flow in the ductile iron pipe sub-DMA in Orleans was also measured on 18 and 19 October 2006 under reduced pressures of ~53 and 71.5 psi, respectively. As can be seen from flow nightlines (Figure 15 and Figure 16), there were several long enough periods over which minimum flow rate remained nearly constant. These rates were considered to correspond to background leakage and were equal to 1.65 and 2.26 L/connection/h at 53 and 71.5 psi, respectively. It should be noted that at these reduced pressures, oscillation over the constant flow periods was significantly less than that under normal operating pressure. The small oscillation at reduced pressure is mainly due to quantization error related to the limited pulse output of the flow meter. The higher the pulse output the lower the flow oscillation.

Cast iron pipes

Night flow was measured on preliminary basis in the cast iron pipe sub-DMA in Meadowlands on 14 November 2006. Sounding of boundary valves revealed 4 noisy valves. Exercising quietened valves V272 and V036 but V294 and V282 remained noisy. The latter valves were also noisy when fully open, which was considered to indicate that there was a leak nearby. This was confirmed in December; when it was also realized that for V294 to be tightly closed in order to stop water flow at the nearby leak it had to be turned down fully and then backward a couple of turns. However, this was not done during flow measurement on the night of 14 November 2006. Therefore, the corresponding flow nightline (Figure 17) of the cast iron pipe sub-DMA with V294 on its boundary may not be representative.

As can be seen from flow nightline in Figure 17, there were several long enough periods over which minimum flow rates, considered equal to background leakage rates almost remained constant at ~0.5 L/connection/h under a pressure of 60 psi and ~6.25 L/connection/h under a pressure of 85 psi (note: pressures were measured at the inlet only because gauges froze at other exposed locations). The minimum flow rate at ~85 psi is close to estimates obtained using current practice for distribution systems in average condition. However, unlike results obtained for the ductile iron pipe sub-DMA in Orleans, the above background leakage rates deviated substantially from the expected leakage-pressure power relationship. This was almost certainly caused by suspected passing valves V294 and V282.

Subsequently the boundary of the sub-DMA was modified by adding valve V275 to the boundary to exclude valves V294 and V282. This reduced the number of service connections in the isolated area to 175. Flow was re-measured on 15 November 2006. As can be seen from the corresponding flow nightline in Figure 18, there were several long enough periods over which minimum flow corresponding to background leakage almost remained constant at ~0.24, 0.96 and 5.31 L/connection/h under inlet pressures of ~49, 62 and 79 psi. These rates are relatively close to the values obtained on 14 November. Hence, it appeared that the significant deviation from the expected leakage-pressure relationship was not only due to suspected passing boundary valves V294 and V282. Additional flow measurements were undertaken in spring 2007 to investigate the cause.

Further measurements of night flow in the cast iron pipe sub-DMA in Meadowlands were performed starting on the night of 10 April 2007. This was exactly the same sub-DMA for which night flow was measured on 15 November 2006 (i.e., the purple area in Figure 31, excluding problematic valve No. V294). The sub-DMA was isolated by closing valves V137, 139, 036, 032, 129, 188, 190 and 275 in addition to permanently closed valves V136, 155, 174 and 202 on the boundary of the Meadowlands high pressure zone. In total, there were 12 boundary valves.

Flow rate early in the night was ~15 L/minute and pressure at the inlet was ~90 psi. When the inlet PRV was adjusted to reduce pressure to 50 psi in order to check the tightness of boundary valves, the flow meter came to complete stop and inlet pressure after the PRV remained at 90 psi. This was considered to indicate a passing boundary valve(s). All boundary valves were then sounded using both an electronic listening device placed on top of a long solid valve key resting squarely on valve nuts. This exercise took all night to complete and at the end all boundary valves were deemed quiet. Passing valve(s) could not be detected probably due to the absence of a significant pressure differential across valves with the high-pressure zone, which is needed to induce noise by passing valves.

On the night of 11 April 2007, pressure in the sub-DMA was reduced to 50 psi by drawing water from fire hydrant H017, near the inlet of the test area, at a high-enough flow rate while partially restricting flow at the inlet. Only boundary valves with the high-pressure zone were sounded. Noise was detected at valves V275, V129 and V036. These were then exercised and re-shut tightly. If noise persisted for a particular valve, it was turned back slightly (a fraction of a turn or more). This helped prevent water from passing over a noisy valve's gate. Subsequently, no noise could be detected at the above valves and it became possible to reduce pressure in the test area to 50 psi via the inlet PRV.

Pressure was logged only at the inlet of the test area and it was monitored at the highest pointing in the test area (fire hydrant H080) using a dial gauge. When pressure was 90, 64 or 48 psi at the inlet, it was 80, 58 or 46 psi, respectively, at the highest point. The narrowing gap between pressures at the inlet and the highest point as the pressure in the test area became lower should have been taken as indication of a passing boundary valve(s). In later tests, after the boundary was altered and re-proved on 18 April 2007, the gap between pressures at the inlet and highest point remained nearly constant regardless of pressure. When pressure was 85, 64 or 50 psi at the inlet on 19, 24 and 23 April 2007, it was 72, 51 or 37 psi, respectively, at the highest point. Once pressure was set to a particular value, it varied insignificantly both at the inlet and the highest point, as water demand was low and reasonably steady. As an approximation, average zone night pressure (AZNP) was taken as the average of pressures at fire hydrant H080 (highest point) and the inlet (lowest point), corrected for pipe depth from the elevation of pressure sensors (~3 metres). In other words, AZNP was determined as pressure recorded at the inlet minus 6.4 psi plus 4.4 psi.

Flow was measured between approximately 1:00 and 3:00 AM while average pressure was 51 psi and between 3:00 and 4:00 AM while pressure was 68 psi. As can be seen from the flow nightline in Figure 19, there were several long enough intervals over which minimum flow rate almost remained steady. The minimum rate (average value of the minima of several intervals) was 0.47 and 0.62 L/connection/h at 51 and 68 psi, respectively. As for rates measured in November 2006, these background leakage rates are much lower than the values of 1.31 and 2.02 L/connection/h at 51 and 68 psi, respectively, predicted by Eq. (2) for a system in good condition having a connection density of 50 connections per km and average service pipe length of 15 m. The N_1 exponent for a power leakage-pressure relationship based on the above measured background leakage rates was equal to 0.962, which is significantly lower than the 1.5 exponent used in current practice.

The large difference between predicted and measured background leakage rates may be due to truly low background leakage rate in the test area. Or it may be due to a valve(s) passing water into the test area from the Meadowlands high-pressure zone at rates of 3.5 and 4.8 L/minute at pressures of 51 and 68 psi in the test area, respectively. This trend was contrary to the expectation that the rate at which water passes through a boundary valve into the test area would be inversely proportional to pressure in the test area (i.e., proportional to the differential of pressures inside and outside the test area). An explanation for this may be that the area of the valve opening through which water passed to the test area increased (i.e., the passing valve became less tight) as the pressure differential across the boundary decreased.

As usual, prior to measuring flow in the test area, all boundary valves were sounded after inducing a pressure differential (between 20 and 35 psi) across valve gates. Although it was certain that a boundary valve(s) was passing, no noise of passing water could be detected at any boundary valve. Possible reasons of this are discussed later.

On the night of 12 April 2007, flow was re-measured in the Meadowlands sub-DMA (purple area) at normal operating pressure and at a reduced pressure of 68 psi to determine if results obtained on the previous night were repeatable. Again, pressure in the test area could not be lowered via the inlet PRV to create a pressure differential across boundary valves so that they could be sounded. Most likely, this was due to one or more boundary valves that were passing water at a significant rate. Subsequently, pressure was lowered to 50 psi by drawing water at fire hydrant H017 inside the of the test area while partially restricting flow at the inlet.

Valves at the boundary with the rest of the Meadowlands high-pressure area were sounded using electronic listening equipment attached to a solid valve key lowered onto valve nuts. Permanently closed valves were not sounded as they were already proved on the previous night. Noise was detected at valves V275 and V036 and it was eliminated by slightly turning the valves backwards (a fraction of a turn). Subsequently, water draw from hydrant H017 was stopped and pressure in the test area increased to an estimated AZNP of 87 psi. Flow was measured using a ¾-inch Neptune T-10 PD meter. The inlet PRV was bypassed, as water flow was high and led to significant pressure loss in the PRV. For comparison, pressure was reduced to an estimated AZNP of 68 psi at the end of the night from about 3:30 to 4:30 AM.

As can be seen from the flow nightline in Figure 20, there were several long enough periods that had a slightly fluctuating minimum value with an average of 3.6 L/connection/h at 87 psi. The slight fluctuation is believed to be due to minor pressure transients in the network. Pressure fluctuation would have been significantly reduced if pressure inside the test area were controlled via the inlet PRV. The above minimum night flow rate was relatively close to the background leakage rate of 2.92 L/connection/h predicted by Eq. (2) for a connection density of 50 connections per km and average service pipe length of 15 m.

It can also be seen from Figure 20 that there were several long enough periods over which the minimum flow rate was steady at ~0.62 L/connection/h at 68 psi. This is the same as the value obtained at the same pressure on the previous night, and is significantly lower that the value of 2.02 predicted using Eq. (2). The N_1 exponent corresponding to the above measured background leakage rates at 87 and 68 psi was 7.14, which is much higher than 1.5 exponent used in current practice. Unfortunately, the above results did not provide a consistent trend with results obtained on the night of 11 April 2007. Measured flow rates and the very high N_1 exponent may be real; or one or more boundary valves were passing.

To further investigate underlying causes, the cast iron pipe test area was significantly enlarged to an area comprising the test area used on 11 and 12 April 2007 (purple area in Figure 31) and an area of similar size (blue area in Figure 31). The following valves were temporarily closed to isolate the enlarged sub-DMA: V087B, 137, 302, 287, 296, 294, 282, 190, 188, 141, 129, 036, 032. The following permanently closed valves were part of the boundary of the sub-DMA: V136, 138, 155, 174, 202, 266, 319 and 309. Total number of valves that formed the boundary of the enlarged sub-DMA was 21. Total pipe length was 5.75 km of which 19.2% was PVC, 7.7% was ductile iron and 73.1% was cast iron pipe. Total number of services was 392 and they included a restaurant, an office building and a school (all of which were unoccupied during flow measurements).

Flow was measured in the enlarged area on the night of 17 April 2007. It was not possible to reduce pressure in the enlarged sub-DMA using the inlet PRV. Suspecting that boundary valve V294 on Meadowlands Drive was passing, it was back turned two turns. Subsequently, it was possible to reduce pressure at the inlet of the sub-DMA to ~50 psi using the PRV. Only valves V294, 129, 302, 287, 137 and 296 were sounded. There was very faint noise at isolation valves V137 and 296 that was not believed to correspond to noise typically induced by passing water. Valves

V190, 188, 141, 032 and 036 were not sounded as they were always found to be quiet when sounded during the previous week. Permanently closed boundary valves of the blue zone V266, 309 and 319 were checked on 1 May 2007; no noise was detected.

Pressure in the sub-DMA was maintained at an estimated AZNP of 53 psi and flow was logged starting at 1:00 AM. It can be seen from the flow nightline in Figure 21 that between 1:00 and 2:00 AM there were several long enough periods having a steady minimum flow of ~0.23 L/connection/h. However, after that it was observed that pressure started to increase on its own and flow had stopped. In view of the faint noise heard earlier at isolation valves V137 and V296, it was suspected that check valves CV293 and CV138 had opened slightly and water started passing into the sub-DMA. V137 and 296 were sounded again but there was only very faint noise that again was not believed to correspond to noise typically induced by passing water. Just in case this was not so, valve V139 was closed to double isolate check valve CV138 but this did not help reduce pressure in the sub-DMA.

Unable to determine what happened, pressure in the sub-DMA was raised to operating level by slowly bypassing the PRV at the inlet. When the PRV was fully bypassed, AZNP reached an estimated value of ~90 psi. As can be seen from the flow nightline in Figure 21, there were a few long enough periods that had a slightly fluctuating minimum value with an average of ~4 L/connection/h. As with previous results obtained for the smaller sub-DMA, background leakage rate at AZNP of ~90 psi was much higher than that obtained at AZNP of 53 psi. In view of this, it's very likely that the large unexpected difference between background leakage rates at different pressures was due to one or more passing valves. Most likely, while AZNP was ~53 psi, the gate of valve V294 moved somehow after it was adjusted prior to flow measurement, it started passing water again and subsequently pressure in the sub-DMA increased. This was confirmed at ~4:00 AM by the failure to reduce pressure in the test area below 82 psi via the inlet PRV. Valve V294 was then sounded and found noisy; it became quiet after it was back turned slightly. Subsequently, pressure in the sub-DMA started to drop (inlet PRV remained set to

~50 psi). However, after few minutes, V294 became noisy again and pressure in the test area started to increase. It became certain that this valve was unreliable.

The boundary of the enlarged sub-DMA was subsequently altered by including valve V273 in order to exclude V294. This reduced the number of service connections by 40 services to 352. Flow in the altered sub-DMA was measured on 18 April 2007. To check boundary valves, pressure was reduced to an AZNP of 47.5 psi. Permanently closed valves were checked on previous nights and it was not necessary to re-sound them, as they were not operated since the previous check. Remaining boundary valves were sounded while flow was being logged at an AZNP of 47.5 psi. At ~1:23 AM, noise was detected at valve V032, which was subsequently eliminated by turning the valve backward by ~1 turn from the fully shut position. At ~3:20 AM, pressure in the test area was increased by slowly opening the main bypass valve of the inlet's flow meter/PRV. After normal operating pressure was reached, the main bypass valve was closed. Flow passed only through the flow meter; the PRV was bypassed because flow was too high and led to significant pressure loss.

It can be seen from the flow nightline in Figure 22 that prior to fixing V032, there were at least two long enough periods over which minimum flow was steady at 0.24 L/connection/h at an estimated AZNP of 49.3 psi. After fixing V032, minimum flow over two long enough periods rose to 1.44 L/connection/h at an estimated AZNP of 48.8 psi. This was reasonably close to the leakage rate of 1.18 liters per connection per hour obtained from Eq. (2) used in current practice, for connection density of 68 connections per km and average service pipe length of 15 metres.

It's interesting to note that background leakage rate prior to fixing V032 at ~1:23 AM was almost the same as that measured on the previous night at almost the same pressure. V032 was sounded on previous nights but noise could not be detected. This could be due to the fact water was passing at an undetectable low rate prior to almost doubling the size of the test area.

It can also be seen from Figure 22 that after pressure was raised to normal operating level, there were at least two long enough periods over which minimum flow rate fluctuated slightly around ~2.97 L/connection/h at estimated AZNP of 85.2 psi. Leakage rate predicted by Eq. (2) was 2.7 litre per connection per hour for connection density of 68 connections per km and average service pipe length of 15 metres. The leakage-pressure power exponent N_1 corresponding to measured leakage rates at 48.8 and 85.2 psi after valve V032 was fixed was 1.3.

Flow measurements with no pressure stepping were repeated in the enlarged sub-DMA at AZNPs of ~82.5, 48.5 and 62 psi on 19, 23 and 24 April 2007, respectively. Prior to each measurement, pressure in the test area was first reduced to ~50 psi, non-permanently closed boundary valves were sounded, noisy boundary valves were adjusted as needed, then pressure was set to one of the above values.

It can be seen from flow nightlines in Figure 23, Figure 24 and Figure 25 that there were several periods during each pressure setting over which minima were relatively steady. These were 1.44, 3.5 and 3.12 L/connection/h at estimated AZNPs of 49.6, 62.5 and 85.7 psi, respectively. Corresponding background leakage rates predicted by Eq. (2) for a connection density of 68 services per km and average service pipe length of 15 metres for a system in good condition were 1.19, 1.69 and 2.71 L/connection/h. At 49.6 and 85.7 psi, measured background leakage rates are reasonably close to levels measured on the night of 18 April 2007 and to levels predicted by Eq. (2). The leakage-pressure power exponent N_1 corresponding to measured leakage rates at 49.6 and 85.7 psi was 1.41, compared to 1.3 obtained from results of flow measurement on the night of 18 April 2007. The average of these two N_1 exponents is 1.35, which is slightly lower than the 1.5 exponent used in current practice.

At 62.5 psi, measured background leakage rate was higher than that measured at 85.7 psi and is more than twice the rate predicted by Eq. (2). The difference between predicted and measured leakage rates at 62.5 psi corresponds to a loss of ~9 litres per minute. This could be due to a tap that was left open at the school in the test area or a toilet that was left leaking (e.g., handle of its tank was not jiggled to stop the leakage). To verify the rate of background leakage at 62.5 psi, flow in the enlarged sub-DMA was re-measured on the following two nights.

On 25 April 2007, flow was re-measured at a reduced AZNP of 63.5 psi. As usual, prior to flow measurement, temporarily closed boundary valves were sounded at reduced pressure of ~50 psi and adjusted as needed. It can be seen from the flow nightline in Figure 26 that initially there were two long enough periods (at 1:34 and 1:47 AM) having a steady minimum flow of 3.6 L/connection/h at an estimated AZNP of 65.2 psi. This was very close to the rate of 3.5 L/connection/h measured at 62.5 psi on the previous night. Then, the minimum flow rate increased to 6.1 L/connection/h at estimated AZNP of 64.7 psi over several long enough periods between 1:48 and 3:25 AM. At 3:30 AM, the inlet PRV was bypassed and pressure in the test area increased to normal operating pressure. It can be seen from Figure 26 that at 3:55 AM flow rate reached a steady minimum rate of 8 L/connection/h over a long enough interval at an estimated AZNP of 78.5 psi. This increase in background leakage rate from the previous rate of 6.1 L/connection/h at 64.7 psi corresponds to a power exponent N_1 of 1.4.

While pressure in the test area was still at normal operating level, minimum flow rate dropped to a steady rate of 6.35 L/connection/h over a long enough interval at 4:13 AM at an estimated AZNP of 81.5 psi, as can be seen in Figure 26. At ~4:20 AM, pressure in the test area was reduced again to an AZNP of 63.5 psi and at 4:23 AM the flow rate reached a steady minimum of 4.8 L/connection/h at an estimated AZNP of 64.4 psi. This decrease in background leakage rate from the previous rate of 6.35 L/connection/h at 81.5 psi corresponds to a power exponent N_1 of 1.19, which is relatively close to the exponent obtained for the previous pressure step. At 4:45 AM, while pressure was at 67 psi, flow rate reached a lower steady minimum rate of 3.7 L/connection/h, which is almost the same as the rate observed earlier in the night.

On 26 April 2007 flow was first measured under an AZNP of 63 psi. From the flow nightline in Figure 27, it can be seen that there were several long enough intervals with a steady minimum flow rate of 3 L/connection/h at an estimated AZNP of 64 psi. At ~3:50 AM, pressure in the test area was stepped down to an AZNP of 48 psi and subsequently the flow rate reached a steady minimum rate of 2.16 L/connection/h at an estimated AZNP of 49.3 psi over a long enough interval. This decrease in background leakage rate from the previous rate of 3 L/connection/h at 64 psi corresponds to a leakage-pressure power exponent of 1.26.

At ~4:15 AM, pressure was slowly stepped up to normal operating pressure by bypassing the inlet PRV. Subsequently, the flow rate reached a steady minimum rate of 4.31 L/connection/h at an estimated AZNP of 84.4 psi over a long enough interval. This increase in background leakage rate from the previous rate of 2.16 L/connection/h at 49.3 psi corresponds to a leakage-pressure power exponent of 1.29. This exponent is reasonably close the one obtained from the previous pressure step and the exponents of 1.4 and 1.19 obtained from pressure steps on the previous night. The average of these four N_1 exponents is equal to 1.3. This is close to the average exponent of 1.35 obtained from much lower background leakage rates measured on 18, 19 and 23 April 2007.

As seen from the above results there was significant fluctuation in background leakage rate from night to night, and sometimes over the same night, that did not correspond to pressure stepping. From field measurements and predictions using N_1= 1.35 and 1.4 for lower and upper range limits, respectively, the range of background leakage rate was 1.44-4.2, 2.1-6.1 and 2.97-8.97 L/connection/h at 49.6, 64.7 and 85.2 psi, respectively.

Residential Night Demand

Average residential night demand obtained from flow measurements in the ductile iron pipe sub-DMA in Orleans and the cast iron pipe enlarged sub-DMA in Meadowlands are presented in Table 1 and Table 2, respectively. Demand rates are presented for several 1-hour and 2-hour periods between 1:00 and 4:30 AM, as well as for the whole 1:00 to 4:30 AM period. Moving 60-minute average residential demand based on flow measurements in Orleans and Meadowlands is also presented graphically in Figure 28 and Figure 29, respectively.

Residential demand was calculated by subtracting background leakage rate from the flow rate at the inlet of the particular sub-DMA, averaged over specified time periods. Residential demand determined as such does not include losses from residential plumbing – these are included in background leakage rates.

The following observations can be made based on results in Table 1 and Figure 28 for the ductile iron pipe sub-DMA in Orleans:

1. Except for the night of 17 Oct 2006 between 3:00 and 4:00 AM and the night of 19 Oct 2006 between 2:00 and 4:00 AM, residential demand was always higher than the estimate of 1.7 L/household/h used in current practice based on flow measurements in the 1990s in the U.K. and Germany (Report E, 1994). This was most pronounced for demand measured during the lawn-watering season in June, which was up to 3 times the rate used in current practice. For a 2000 service connection DMA, this could lead to a spurious recoverable leakage rate of up to 113 litres per minute, which is equivalent to about 4 service pipe leaks or a break of a small-diameter distribution pipe.

2. Residential night demand varied significantly from night to night in June, most likely as a result of lawn watering at night. For example, demand on the night of 28 June was higher than that on the nights 26 or 27 June by up to 2.75 L/household/h. It rained heavily during the nights of 26 and 27 June and hence fewer (if not none) water sprinklers were in use.

3. In October, which falls outside the lawn-watering season, residential demand varied only slightly from night to night over the same 2-hour period. The demand was generally higher than the 1.7 L/household/h used in current practice. However, it was seldom outside the 95% confidence

range of 1.79 to 2.89 L/household/h. This range was determined based on the assumption that in any hour of the night, average demand is from a proportion "*p*" out of total number of residents "N_r". Subsequently, the statistical distribution of the nightly number of active residents is Binomial, for which the mean is given by (Report E, 1994):

Eq. (4): $\mu_{active} = p \cdot N_r$

and the standard deviation is given by:

Eq. (5): $\sigma_{active} = \sqrt{p \cdot (1-p) \cdot N_r}$

Assuming that active residents have equal demand of "*z*" litres per resident per hour, the number of people per residence is n_r and the number of service connections is "N_c", then the mean and standard deviation of night demand per connection per hour is given by:

Eq. (6): $\mu_{demand} = z \cdot p \cdot n_r$

and

Eq. (7): $\sigma_{demand} = z \cdot \sqrt{p \cdot (1-p) \cdot n_r / N_c)}$

The above 95% confidence range, i.e., $\mu_{demand} \pm 2\sigma_{demand}$, was obtained by assuming that 6% of people are active in any night hour; an average of 3 people per residence; a demand of 13 litres per resident per hour (corresponding to water volume used by a toilet flush); and total number of services equal to 298.

4. In October, i.e., outside the lawn-watering season, overall average residential demand during the 1:30-3:30 AM and 2:00 to 3:00 AM periods was equal to 2.08 and 2.03 L/household/h, respectively.

5. Demand measured on the rainy nights of 26 and 27 June during 1:30-3:30 AM and 2:00-4:00 AM was higher than corresponding demand in October but it also generally fell within the above 95% confidence limits. A higher

proportion of people can be expected to be awake at night during the summer months (June to August). Hence, even if no sprinklers were in use on some nights, residential demand during summer months could be expected to be higher than during the rest of the year.

6. Generally, demand decreased over the same night. In October, overall average demand decreased from 2.7 L/household/h during 1:00-3:00 AM to 1.85 L/household/h during 2:00-4:00 AM; and it decreased from 3.36 L/household/h during 1:00-2:00 AM to 1.66 L/household/h during 3:00-4:00 AM.

7. During preliminary flow measurement on 7 June (not listed in Table 1), average flow rate at the sub-DMA's inlet was ~10.25 L/household/h at ~74 psi pipe average pressure. Background leakage rate at this pressure was equal to ~2.37 L/household/h (see relationship in Figure 33 derived later). Subsequently, actual residential demand was equal to 7.97 L/household/h. This is almost 5 times the currently used value of 1.7 L/household/h and if not accounted for would lead to a false recoverable leakage rate of ~200 litres per minute for a DMA having 2000 service connections. For the whole distribution pipe network in Ottawa, an error in residential demand of ±1 L/household/h corresponds to a leakage rate of ±1.16% (assuming 165,000 service connections and water production of 340 ML/day). Therefore, for reliable estimation of leakage rates under the condition of variable night demand, it may be necessary that residential demand be measured using AMR synchronously with water flow into DMAs.

For the cast iron pipe sub-DMA in Meadowlands (purple and blue zones), the following observations can be made based on results in Table 2 and Figure 29:

1. In general, residential demand was significantly higher than the estimate of 1.7 L/household/h used in current practice. In some instances, it was more than double the currently used estimate.

2. Measured residential demand was generally within the 95% confidence range of 1.77 to 2.91 L/household/h. This range was obtained assuming that 6% of people are active in any night hour; an average of 3 people per residence; a demand of 13 litres per resident per hour (corresponding to water volume used by a toilet flush); and total number of services equal to 352.

3. Residential demand varied slightly from night to night over the same 1- or 2-hour period.

4. Overall average residential demand during the 1:30-3:30 AM and 2:00 to 3:00 AM periods was equal to 3 and 3.2 L/household/h, respectively.

5. Generally, demand decreased over the same night. Overall average demand decreased from 3.58 L/household/h during 1:00-3:00 AM to 2.66 L/household/h during 2:00-4:00 AM; and it decreased from 3.96 L/household/h during 1:00-2:00 AM to 2.1 L/household/h during 3:00-4:00 AM.

Recoverable Leakage

Recoverable leakage rates for the ductile iron pipe DMA in Orleans obtained from flow measurements over six nights in June and October 2006 are presented in Table 3. Leakage rates were obtained from a water balance using the flow rate method by subtracting residential and non-residential demand and background leakage rate from the supply flow rate. Results are presented for flows averaged over the periods 1:30-3:30 AM and 2:00-3:00 AM. For comparison, results are also presented for flows averaged over 2:00-4:00 AM and 2:30-3:30 AM and for the minimum of the moving 60-minute average flow rate. Residential demand was taken equal to 2; 1.8; and 2.34 litres per household per hour over the periods 1:30-3:30 AM and 2:00-3:00 AM; 2:00-4:00 AM and 2:30-3:30 AM; and the minimum 60-minute moving average flow rate method, respectively. A retirement home was the only major non-residential user in the Orleans DMA; its average night demand was determined to be ~20 litres per minute. Background leakage rate was estimated based on the relationship in Figure 33, established later, with average pressure in the corresponding period.

As can be seen from Table 3, there was significant variation in recoverable leakage rate from night to night. This was most pronounced during June, which falls in the lawn-watering and vacation season. Leakage rate did not vary with pressure as expected, i.e., it did not necessarily increase with pressure. For example, the lowest recoverable leakage rate in June occurred on the 19th under the highest pressure of ~89 psi. This inconsistent variation is almost certainly attributable to lawn watering at night since the number sprinklers in use could vary from night to night, depending on precipitation (note: some sprinkler systems may be fitted with moisture sensors). It rained heavily on 19 June and subsequently fewer lawn sprinklers might have been in use in comparison to other nights. Prior to flow measurements, the whole DMA was acoustically surveyed for leaks by city staff but none were detected.

Recoverable leakage rates in the ductile iron pipe DMA in Orleans in late October were lower than those measured in June. Most likely, this was due to the fact that in general lawns are not watered at the time flow was measured in October. Another acoustic leak survey of the DMA by city staff, prior to flow measurements in October, also did not reveal any leaks. The non-zero rates of recoverable leakage obtained in October are most likely due to underestimation of residential demand and (or) variable plumbing leakage in residences and major establishments such as schools, of which there were 4 in this DMA. This could also explain the unexpected variation of recoverable leakage rate with pressure, i.e., its slight increase with decreasing pressure.

Flow in the cast iron pipe DMA in Meadowlands was preliminarily measured on 16 November 2006 but the flow rate was unexpectedly high (it was beyond the high limit of the 2-inch positive displacement meter used). Initially, as a result of routine boundary valve check, the high flow was suspected to be due to a boundary breach at V269 on Merivale Road, which is on the permanent boundary of the

Meadowlands high-pressure zone. City staff checked this and another suspected valve on the permanent boundary and reported they were fully closed.

Subsequently flow was re-measured on 23 November 2006 using a 3-inch turbine meter. As can be seen from the flow nightline in Figure 30, the flow rate was ~750 litres per minute (~33.2 L/connection/h). This is much higher than the expected background leakage rate for a system in average condition and residential night demand equal to ~4 L/connection/h and ~2.34 L/household/h, respectively. Such high flow rate is indicative of a major leakage in the DMA or a breach of its boundary or both. Boundary valves in were re-checked but all were found to be “quiet”. Also, initially city staff sounded all fire hydrants; no leaks were found except for a minor leak at fire hydrant H063 that was subsequently repaired. Then they conducted a correlator-based leak survey of the DMA using a Palmer MicroCorr 6 correlator (vibration sensors were attached to fire hydrants.) Again no leaks were detected.

Subsequently, it was decided to step test night flow in the DMA in order to narrow down the area of suspected leakage. The DMA was divided into 5 areas (Figure 31). The flow step test was performed on the night of 19 December 2006 with the flow monitored between ½ to 1 hour for each step. Step 1 comprised area 1 and step 2 comprised areas 1 and 2, etc. The flow nightline of the step test is shown in Figure 32. Flow increased slightly as the steps progressed up to step 5 when it increased substantially by ~500 L/minute. This indicated major leakage in area 5.

Then Area 5 was thoroughly surveyed for leaks using the LeakfinderRT leak noise correlator (Hunaidi & Wang, 2006). Accelerometers were attached to all fire hydrant pairs in a leap-frog fashion. The survey was conducted on 27 and 28 December 2006 by NRC and city staff. The correlation survey revealed a leak in a 10-inch cast iron pipe between fire hydrants H117 and H118 on Meadowlands Drive, slightly to the east of Eagle Lane. City staff confirmed the position of the leak using ground geophones and chlorine test on a water sample from a nearby storm water manhole. A city crew excavated the suspected leak in early January 2007 and found a full circumferential pipe break. Subsequent to the repair, the minimum night flow rate in the Meadowlands high-pressure zone, of which the cast iron pipe DMA is part, dropped by ~600 litres per minute.

Measurements of night flow in the full Meadowlands DMA were also made on the nights of 16, 19, 30 April and 1 May 2007 at estimated AZNPs of ~81, 47 and 64 psi, respectively. AZNP of 81 psi was the normal operating pressure in the DMA. Reduced AZNPs of 47 and 64 psi were realized by throttling a ball valve after the inlet flow meter. The DMA was isolated between 1:00 and 4:00 AM. Flow was measured using a 2-inch Neptune T-10 positive displacement flow meter.

Prior to flow measurements in April and May 2007, the DMA was surveyed again for leaks by city staff but no leaks were detected. NRC staff also correlated noise at all fire hydrants in the zone west of Merivale Road, which was not included in the flow step test conducted in December 2006; also no leaks were detected. All boundary valves were re-sounded; noise was detected only at permanently closed valve V269 on Merivale Rd. (near Capilano St.). Noise at this valve was initially detected in November 2006; it was reported to the city staff back then but it's not certain how it was fixed. Noise was also reported to city staff after it was detected on 16 April 2007. However, after valve V269 was reported fixed, noise could still be heard at it (both when shut tightly or back turned slightly). An attempt to alter the boundary of the DMA to exclude V269 by closing V020, 122 and 123 on Merivale Road did not succeed because V123 could not be located (most likely it's paved over.)

Correlation of leak noise in December 2006 at nearby fire hydrants H050 and H051 on Capilano St. and in April 2007 at fire hydrants H049 and H008 did not reveal an out-of-bracket peak corresponding to noise at V269. Therefore, noise that could be detected at this valve is most likely due to a leak outside the DMA. Noise from such a leak would be reflected back at the permanently closed V269 valve and hence it could not be correlated at points inside the DMA. A search for the suspected leak by sounding valves V021 and 023 is recommended.

Results of flow measurements on the nights of 16, 19 and 30 April and 1 May 2007 are presented in Table 4. Recoverable leakage rates were obtained from a water balance using the flow rate method by subtracting residential demand and background leakage late from the supply flow rate. For comparison, recoverable leakage rates excluding toilet leakage are also presented. Results are presented for flows averaged over the periods 1:30-3:30 AM and 2:00-3:00 AM. For comparison, results are also presented for flows averaged over 2:00-4:00 and 2:30-3:30 AM and for the minimum of 60-minute moving average flow rate. Residential demand was taken equal 3; 2.7; and 2.34 L/household/h over the periods 1:30-3:30 and 2:00-3:00; 2:00-4:00 and 2:30-3:30; and for the minimum 60-minute moving average flow rate method, respectively. There were no major active non-residential users in the Meadowlands DMA and therefore non-residential demand was assumed to be negligible. Background leakage was estimated based on the relationship in Figure 35 (presented later) with average pressure in the corresponding period. Toilet leakage was estimated based on the relationship yielding maximum rate in Figure 37 (also presented later) with average pressure in the corresponding period.

The following observations can be made based on the results in Table 4 for the Meadowlands DMA:

1. There was significant total recoverable leakage (equal to about or greater than 100 litres per minute) at all AZNPs although no leaks could be detected in the DMA using acoustic surveys.
2. Total recoverable leakage varied by ~30 litres per minute between the nights of 16 and 29 April 2007, which had the same AZNP of ~81 psi. However, it had almost the same rate on the nights of 30 April and 1 May, which had AZNPs of ~47 and 64 psi, respectively. The leakage-pressure power exponent, N_1, varied between 0 and 3. This may be considered to indicate a large variation of mainly toilet leakage from night to night, as residential demand measured in this area varied only slightly from night to night.

3. Recoverable leakage excluding toilet leakage also varied inconsistently from night to night. The leakage rate was almost negligible (less than about 20 litres per minute) on the nights of 16 April and 1 May at ~81 and 64 psi, respectively. However, recoverable leakage was considerably higher on the nights of 29 and 30 April at ~81 and 47 psi, respectively. Since no leaks could be detected in the DMA, recoverable leakage on the nights of 29 and 30 April may be taken as indication that toilet leakage was underestimated. Background leakage in the Meadowlands sub-DMA was not monitored over many nights and subsequently maximum possible toilet leakage could have been missed. For recoverable leakage at ~47 psi AZNP on 30 April to be negligible, maximum toilet leakage rate should be ~4.7 litres per household per hour at 47 psi (compared to the assumed rate of 2.2).
4. There was only a small difference between recoverable leakage rates obtained using flow rates averaged over 1:30-3:30 AM, 2:00-3:00 AM, 2:00-4:00 AM, and 2:30-3:30 AM. However, as expected, leakage rate obtained using minimum of the 60-minute moving average flow rate was always lower than leakage rates obtained using average flow rates over the above fixed periods.

Leakage-Pressure Relationship

It's generally assumed in practice that leakage rate in water distribution networks varies with pressure to the power N_1, with the power exponent possibly ranging from 0.5 to 2.5, depending on the type of pipe material and type and size of leaks. According to Lambert (2001), small background leaks in both metal and plastic pipes are very sensitive to pressure with N_1 being close to 1.5; large detectable leaks in plastic pipes have N_1 equal to 1.5 or higher; and large detectable leaks in metal pipes have N_1 close to 0.5.

For background leakage measured in the Orleans sub-DMA consisting of ductile iron pipes, N_1 was ~1.11 for the best-fit power relationship in Figure 33. This exponent is significantly lower than the 1.5 exponent suggested by Lambert (2001). For cast iron pipes, based on preliminary measurements, N_1 was initially ~6.5 for best-fit power relationship in Figure 34. This exponent is very high and, based on subsequent extensive flow measurements in spring 2007, it was determined that the cause was an acoustically undetectable passing boundary valve (V032).

As discussed earlier, there was significant fluctuation in background leakage rate measured in spring 2007 in the Meadowlands area, comprised mostly of cast iron pipes. Fluctuation was from night to night and sometimes over the same night and it did not correspond to pressure stepping. The cause of this unexpected fluctuation is not certain but it could be attributed to variable plumbing losses, especially toilet leakage. The variation of background leakage rate versus pressure measured on the nights of 18, 19 and 23 April 2007 is presented in Figure 35. These leakage rates are not believed to include significant leakage from toilets because they were reasonably close to rates predicted by Eq. (2), which was based on U.K. data ascertained to be toilet leakage free (Report E, 1994) and (Lambert, 2008). Best-fit power relationship between pressure and leakage rate in Figure 35 had N_1 equal to ~1.35, which is slightly lower than the 1.5 exponent suggested by Lambert (2001) for background leakage.

Variation of background leakage rate versus pressure measured in the Meadowlands mostly cast iron pipe sub-DMA on the nights of 25 and 26 April 2007 is presented in Figure 36. Unlike background leakage rates measured on the nights of 18, 19 and 23 April 2007, leakage rates in Figure 36 are believed to include leakage from toilets because they are significantly higher than rates for a system in good condition predicted by Eq. (2), which as mentioned earlier is based on U.K. data ascertained to be toilet-leakage free. The N_1 exponents of best-fit pressure-leakage relationships in Figure 36 varied from 1.19 to 1.4 with an average of ~1.3.

Variation of toilet leakage rate with pressure in Meadowlands cast iron sub-DMA is presented in Figure 37. Toilet leakage was estimated as total background leakage measured on the nights of 25 and 26 April 2007 minus background leakage estimated using the power relationship in Figure 35 that was fitted to minimum flow rates measured on the nights of 18, 19 and 23 April 2007. The pressure-toilet leakage power exponents N_1 in Figure 37 varied from 1.06 to 1.43 with an average of ~1.2.

Leak detection surveys in the ductile iron pipe DMA in Orleans did not uncover any leaks and therefore it was not possible to determine a relationship between recoverable leakage rate and pressure. The spurious recoverable leakage rate detected on the nights of 20 to 22 June 2006 in this DMA was most likely due to the use of lawn-watering sprinklers. The corresponding N_1 was ~1.5 (Figure 38), which may not be unrealistic for water sprinklers not fitted with pressure regulators.

Also, it was not possible to measure the night flow rate versus pressure in the cast iron DMA in Meadowlands before the repair of the major leak on Meadowlands Drive. Normal operating pressure at the critical point in the DMA was 45 psi, which was near the minimum pressure allowed by city staff. For recoverable leakage rates measured in spring 2007 after the major leak on Meadowlands Drive was repaired (Table 4), N_1 varied erratically from about 0 to 3.3, most likely because of variable significant toilet leakage.

Analytical Identification of Flow Components

Analytical identification of the components of minimum moving 60-minute average flow rates measured on the nights of 20 to 22 June 2006 was first attempted assuming N_1 = 1.5 for background leakage and 0.5 for recoverable leakage, which are used in current practice. Residential demand was assumed to be constant. Minimum moving 60-minute flow rates were 7.4, 8.6 and 10.4 L/connection/h under average pipe pressures of 48.4, 60.6 and 72, respectively (note: pipe pressures used earlier by Hunaidi & Brothers (2007b) were not corrected for pipe depth of ~ 3 metres below the level of pressure sensors at fire hydrants.) This led to background leakage rate, recoverable leakage rate and residential demand of 19.3, -31 and 22.5 L/connection/h, respectively, which are significantly different from measured rates. A second attempt was made using measured N_1 exponents of 1.11 and 0.5 for background and recoverable leakage rates, respectively. This led to background leakage rate, recoverable leakage rate and residential demand equal to –40.6, 30 and 12.1 L/connection/h, respectively. These are also significantly different from measured rates.

7 DISCUSSION

Background Leakage

There was significant fluctuation in background leakage rate from night to night, and sometimes over the same night, that did not correspond to pressure stepping in the Meadowlands area. The range of background leakage rate was 1.44-4.2, 2.1-6.1 and 2.97-8.97 L/connection/h at 49.6, 64.7 and 85.2 psi, respectively. The cause of this wide range is not certain. Pressure during flow measurements fluctuated only slightly and this did not account for the observed large fluctuation of minimum night flow rate.

Also, the boundary of the test area was proved on each night prior to flow measurement, so the likelihood of water passing into or out of the test area was remote. Residential demand determined as average minus minimum supply flow rates was in the expected range on every night when flow was measured. Furthermore, it is unlikely that the observed fluctuation in background leakage was due to residential demand from say faucets that were left running because the observed leakage-pressure power exponent was ~1.3, while that for running faucets is expected to be 0.5.

As noted earlier for the procedure used in this study, background leakage rate was taken equal to the minimum rate of the flow nightline of a small test area after all detectable leaks in the area were repaired. Subsequently, background leakage rate determined as such included losses from distribution system components as well as from residential plumbing fixtures, especially toilets. Leakage from toilets can be caused by a number of problems (Vickers, 2002): (i) a deteriorated flapper valve that does not properly seat in order to tightly seal the tank's drain hole, thus causing water to constantly leak into the toilet bowl; (ii) a worn out refill valve that does not completely shut off after the tank is filled, thus causing water to constantly run over the top of overflow drain tube; (iii) deteriorated float ball and rod causing the refill valve to remain open; (iv) worn out lift chain and handle rod causing the flapper valve to improperly seat against the tank's drain hole; and (v) poorly sized or poorly designed replacement parts such as easily twistable float ball rods that lead to continuously open refill valves or poorly designed lift chains that can be easily tangled leading to improperly seated flapper valves.

Mayer et al. (1999), based on extensive measurement of water use by 1,188 households in the United States and Canada, reported that a significant portion of residential water use was attributed to leakage. The average amount of water lost through leakage in households was 3.45 L/household/h. This is significant in comparison to average legitimate residential night demand and background leakage of distribution systems in good condition. The standard deviation of residential leakage was a high 8.53 L/household/h, indicating wide spread in the data. About 10% of the households studied were responsible for 58% of the leakage found. For these households, Mayer et al. (1999) reported an average leakage of 14.25 L/household/h, primarily due to leaky flapper valves of toilet tanks. Water loss from worn out components of the refill mechanism of toilet tanks (i.e., ball cock, float ball or level) is usually very low and is not captured by residential flow meters. Hence, the actual toilet leakage rate can be even higher than that reported by Mayer et al.

It's not clear if the high standard deviation of residential leakage reported by Mayer et al. (1999) was due to variation of toilet leakage from household to household or due to variation of toilet leakage from night to night in households or both. As noted earlier, toilets can leak at the flapper or the refill valves, but either way the refill valve operates and the flow rate through the refill valve is related to pressure in the distribution system. For flow measurements in the Meadowlands test area, pressure remained nearly constant during each pressure step. Therefore, it may be argued that the rate of toilet leakage should be expected to also remain nearly constant. However, for this to be true, the opening of the refill valve of a leaking toilet must retain the same area between tank flushes. This may be unlikely for aged and deteriorated toilets.

Furthermore, for a leaking flapper valve, it's understood that the refill valve eventually opens after sufficient water has leaked through the flapper valve. After the tank is refilled, the refill valve closes. Water continues to leak though the flapper valve and the refill valve's open/close cycle also continues. Even if it can be correctly assumed that the opening of the refill valve retains the same area from one cycle to the next, total leakage from all toilets with faulty flapper valves will fluctuate over night as the open/close cycles of their refill valves cannot be expected to be in phase (or in sync).

Finally, residents may try to temporarily stop toilet leaks that are visible or audible by "jiggling" the tank's handle before they eventually undertake proper repair. This action can help to untangle the lift chain or nudge the flapper valve into proper seating position. The success of this process may be inconsistent and residents may try it only sometimes. This may lead to fluctuation of toilet leakage from night to night and even over the same night.

To address the difficulty posed by random plumbing losses, Fanner and Harris (undated) suggested that water flow into the test area be monitored over a long-enough period of say 30 nights. The position at which the statistical distribution of the minimum night flow rate tails off would then be indicative of insignificant random plumbing losses and usage. Background leakage is estimated as the mean minus 2.5 to 3 times the standard deviation of the minimum flow rate. Unfortunately, this background leakage rate can still include plumbing losses that remain constant over the monitoring period, e.g., visually and audibly undetectable toilet leakage. Also, Fanner and Harris assumed that the leakage rate remains constant over a 30-day monitoring period, which may be unlikely especially for old systems.

Furthermore, even if the mean of plumbing losses were truly represented by 2.5 to 3 times the standard deviation of the minimum night flow rate, this mean may vary from area to area in the distribution system, depending on the predominant type and vintage of plumbing fixtures. Subsequently, estimated residential night demand to be used in night flow analysis would be inaccurate and the resulting estimate of recoverable leakage would be unreliable.

For systems having residential waters meters fitted with automated meter reading devices (AMR) that can be used to read residential demand synchronously with the reading of DMA flow meter, recoverable leakage determined from a water volume balance may still include toilet leakage. This is because the flow rate of visually or audibly undetectable plumbing leaks is usually very low and is not captured by residential flow meters.

Components of background leakage, e.g., from distribution pipes versus service connection pipes before and after curb stops, could not be measured separately in this study. This is because it was not feasible to close curb stops of service connections in the whole test area all at once. However, it can be seen from Table 5 that background leakage rates predicted using Eq. (1) or Eq. (2), but with exponents determined in this study instead of the currently used 1.5 exponent, are reasonably close to rates calculated using power relationships obtained in this study. This is remarkable in view of the fact that components of background leakage in Eq. (1) and Eq. (2) were obtained with incomplete data (Report E, 1994).

An even better overall agreement, taking into account flow measurements in both Ottawa and Regina, was obtained using the following values for constants A, B and C in Eq. (2): 24 L/km/h, 1.5 L/connection/h and 0.4 L/connection/h, respectively, and the power exponents determined in this study instead of 1.5 (Table 6). These constants lead to combined communication pipes and distribution mains leakage of 1.8 L/connection/h (assuming service connection density of 80 connections/km). As discussed later, this was the total background leakage measured in the U.K. (Report E, 1994).

Setting constant C to 0.4 L/connection/h corresponds to a background leakage rate in supply pipes that is half the rate reported in Report E (1994) for combined background leakage rate in supply pipes and plumbing fixtures. The remaining 0.4 L/connection/h accounts for plumbing leakage that would occur because of inaudible leaks, e.g., due to faulty flapper valves of toilet tanks. This leakage flows into toilet bowls and is not easily detectable, unlike leaks from faulty refill valves that in U.K. systems would overflow toilet tanks causing water to flow through overflow pipes to the outside of the property, where it was clearly visible, and quickly identified and fixed.

The large number of valves, especially in old areas like Meadowlands, made measurement of background leakage rate difficult. City engineers insisted on using the Meadowlands area in this study because of its poor record of pipe breaks and leaks. Difficulties encountered in this area were due to old boundary valves that were passing but could not be detected acoustically when the test area was small, e.g., 182 service connections. The reason behind this was not certain. It might be that the flow rate of passing water was too low to create detectable noise. Or it might be that the gates of some worn out valves moved somehow after they were sounded. This could be due to the removal of valve keys that rested on top of valve nuts during sounding or due to the change of differential pressure across valve gates as result of pressure stepping in the test area.

Therefore, background leakage rates determined on the basis of minimum rates of night flow measured in small areas are unlikely to be reliable for old pipes, unless a zero-pressure test is conducted to prove the area's boundary. However, utilities in Canada do not allow such tests because of the regulatory requirement for super chlorination of pipe sections following a pressure drop below a preset threshold (typically 20 psi). Subsequently, it is recommended that tests to determine background leakage rates and their variation with pressure, i.e., N_1 tests, be conducted in as large areas as possible with very few boundary valves. Larger test areas and smaller number of valves lead to higher flows across passing valves, making the latter more detectable acoustically. The test area should not include active major non-residential users and no more than 400 households. Boundary valves should be sounded using an electronic listening device at the highest possible differential pressure that can be induced across valve gates.

It was discovered that one of the boundary valves in the Meadowlands area was "unstable", i.e., after adjusting it prior to flow measurement to ensure it was not passing, it somehow became noisy later during flow measurement! Most likely, loose valve parts moved as pressure inside the test area was stepped to different level. Problem valves should be repaired or replaced or excluded by changing the boundary of the test area. If this is not possible, it may help to check boundary valves of the test area at each pressure level during N_1 tests.

Residential Night Demand

Overall average residential demand in the Meadowlands area was higher than demand in the Orleans area by about 1 L/household/h over the period 1:30-3:30 AM. This may be due to the fact that homes in the Orleans area are newer than those in Meadowlands and subsequently may have water-conserving toilets, e.g., having small tanks or a dual flush feature. Toilets with the latter feature allow for a choice between a 3 or 4-litre flush for liquid waste and a 6-litre flush for solid waste, whereas older toilets have a fixed flush volume of 13 litres (or more).

Consequently, it may be necessary to use DMA-specific residential night demand in order to accurately estimate the rate of recoverable leakage based on analysis of DMA night flow. DMA-specific residential night demand may be measured in sub-DMAs following the procedure utilized in this study. Alternatively, it may be estimated based on the percentage of active residents in any night hour (6% is used in current practice), average number of residents per household, and demand per resident per hour (corresponding to water volume used by a toilet flush weighted according to type and proportion of toilets). If neither is possible, it is suggested that analysis of night flow be based on DMA supply flow rate averaged over the period from 1:30 to 3:30 or 2:00 to 3:00 AM (instead of minimum of the 60-minute moving average) and average residential demand of 3 and 2 L/household/h for older and newer areas, respectively, (instead of the estimate of 1.7 L/household/h used in current practice).

Recoverable Leakage

It's not clear why the large ~600 L/minute leak in the Meadowlands DMA was initially missed by acoustic surveys of the area. This may be due to faulty equipment. It may also be due to significant leak noise attenuation caused by frozen soil in winter or by the use of short plastic pipe sections or splices for pipe repair or as leads when old fire hydrants are replaced. Plastic pipe sections "muffle" leak noise making it hard to detect. The presence of a deep frost layer during winter, which in Ottawa is usually deeper than one metre, increases the effective mass of fire hydrants and in turn reduces their response to vibration. These difficulties should be factored in acoustic survey strategies. Leak detection equipment should be verified frequently. Also, consideration should be given to the use of hydrophones instead of vibration sensors at fire hydrants or to probably attaching acoustic sensors at inline valves.

Pressure-Leakage Relationship

Results obtained in this study confirmed that variation of background leakage with pipe pressure follows a power relationship. However, especially for ductile iron pipes, the power exponent was less than 1.5 that is used in current practice.

The basis for using an exponent equal to 1.5 for the power relationship between background leakage and pressure is uncertain. It is understood that an exponent equal to 1.5 is easy to interpret theoretically as it corresponds to leaks with opening areas that are linearly proportional to pressure, e.g., joint and longitudinal cracks. However, data presented in the literature concerning background leakage is poorly documented and does not provide a clear evidence of the power exponent being equal to 1.5.

On the other hand, using a much smaller exponent can be easily justified theoretically if it's assumed that water discharge from background leaks is mostly laminar. Subsequently, background leaks would have discharge coefficients that would vary with Reynolds number, e.g., according to the relationship shown in Figure 39 for a very small circular orifice. It can be shown that for Reynolds numbers less than 1000, the exponent for the power relationship between leakage rate and pressure is 0.83.

It was not possible to measure night flow rate at more than one pressure level in the cast iron DMA in Meadowlands before the repair of the major leak on Meadowlands Drive because of an operational restriction on minimum pressure. However, recoverable leakage, which was mostly due to a circumferential pipe break, most likely changed according to a square-root relationship with pressure (i.e., $N_1 = 0.5$). This was the case for recoverable leakage measured under different pipe pressures in an asbestos-cement pipe area in Regina as part of this study, which was also mostly due to a circumferential pipe break.

A square-root leakage-pressure relationship implies that the effective area of leaks does not change with pressure. This is plausible for a circumferential crack, as opposed to longitudinal one. The width of a circumferential crack is primarily governed by soil load. On the other hand, the width of a longitudinal crack is mainly governed by hoop stress in the pipe wall, which is directly related to pipe pressure.

Analytical Identification of Flow Components

Failure to analytically identify night flow components is most likely due to the assumption of constant residential demand, discrepancies between actual and assumed pressure-leakage power exponents and/or use of minimum 60-minute flow rate. The latter was ruled out by using average flow rates between 2:00 and 4:00 AM, which also led to unrealistic flow components.

The system of linear algebraic equations utilized to analytically identify components of night flow is very sensitive to very small variation in residential demand and to a lesser degree to errors in exponents of the leakage-pressure power relationships. For example, consider the case of a system having a total average night flow of 240 L/minute at 82 psi comprised of 40 L/minute background leakage, 135 L/minute recoverable leakage and 65 L/minute residential demand. At 65 psi, the system has a total average night flow of 200.2 L/minute comprised of 28.2

L/minute background leakage, 107 L/minute recoverable leakage and 65 L/minute residential demand. At 50 psi, the system has a total average night flow of 166.4 L/minute comprised of 19 L/minute background leakage, 82.3 L/min recoverable leakage and 65 L/minute residential demand. Subsequently, N_1 is equal to 1.5 and 1.0 for background and recoverable leakage, respectively.

Based on the above total night flows and the assumption that N_1 is equal to 1.5 and 0.5 for background and recoverable leakage, respectively, the following night flow components at 82 psi are obtained analytically: 90.5 L/minute background leakage, 119.6 L/minute recoverable leakage and 29.9 L/minute residential demand. If the actual power exponents of 1.5 and 1 for background and recoverable leakage, respectively, are used but residential demand at 82 psi was 68.23 L/minute i.e., 5% higher than that of the other two measurements, the following night flow components at 82 psi are obtained analytically: -163.7 L/minute background leakage, 408.5 L/minute recoverable leakage and –4.8 L/minute residential demand.

Implication of Equations 1 to 3 on UARL and ILI

Constants B and C in Eq. (1) to Eq. (3) used in current practice are somewhat different from those determined in the original study (Report E, 1994) referenced by Lambert et al. (1999). In Report E (1994), constants B and C were determined to be 1.5 and 0.25 L/connection/h, respectively. The discrepancy in constant B was initially thought to be due to the application of a 20 hour-to-day conversion factor by Lambert et al. (1999), which was commonly used at the time of Report E in the United Kingdom to account for diurnal pressure fluctuation. However, it was clarified that the change from 1.5 to 1.25 L/connection/h was not because of the application of a 20 hour-to-day conversion factor, but because the 1.5 L/connection/h was felt to be not low enough (Lambert, 2008).

The discrepancy in constant C is due to including background plumbing leakage by Lambert et al. (1999), which was specified as 0.5 L/connection/h for a system in average condition in Table 4.1 of Report E (1994) under “supply pipe” loss (i.e., 50% of total background leakage on supply pipes). For a system in good condition, the corresponding value is 0.25 L/connection/h. The reason for including plumbing losses by Lambert et al. (1999) was perhaps the realization that attributing 50% of background losses on supply pipes to plumbing by Report E (1994) was inaccurate. UK plumbing byelaws that were in effect at the time of Report E (1994) did not permit installation of toilet cisterns which would allow water to leak into the toilet bowl – if the float valve assembly leaked, the water would flow through an overflow pipe to the outside of the property, where it was clearly visible, and quickly identified and fixed (Lambert, 2008).

Constants A, B and C determined in Report E (1994) were based on flow measurements in 30 to 52 DMAs in the UK while curbstops were open and then while they were closed. Flow rates from different DMAs were standardized to 71 psi (50 m) pipe pressure using correction factors derived in Report 26 (1980) for “net night flows” comprised of leakage (both background and recoverable) and residential demand. Incidentally, these pressure correction factors may not be accurate when applied to flows that deviate from this definition because of potentially different response of flow components to pressure. Background losses were determined for “supply pipes”, “communication pipes”, and distribution mains. Losses of “supply pipes” comprised losses from plumbing fixtures and service connection pipes beyond the curbstop. Losses from ‘communication pipes” comprised losses from taps on mains, service pipes from mains to curbstops, and curbstops themselves.

Average losses measured for supply pipes were 0.8 L/connection/h; however, this was thought to be on the low side because some people might have not operated automatic washing appliances overnight (Report E, 1994). Subsequently, supply pipe losses were increased to 1 L/connection/h and it was assumed that 50% of this loss was attributable to underground service pipes between curbstops and customer meters, and the other 50% was attributable to residential plumbing.

Average losses measured from distribution mains and communication pipes were 1.8 L/connection/h; however, these also were thought to be somewhat on the low side (Report E, 1994). Subsequently, losses from communication pipes and distribution mains were assessed in Report E to be 3 L/connection/h and 40 L/km/h, respectively. Assuming a connection density of 60 connections per km, assessed losses from distribution mains become 0.67 L/connection/h. Thus total assessed losses from communication pipe and distribution mains were 3.67 L/connection/h, which is twice the measured average of 1.8 L/connection/h.

Further, it was suggested that the above background leakage from supply pipes were indicative of average rates but they "could vary by at least ±50% depending upon the condition of fittings and joints" (Report E, 1994). In other words, for a system in good condition, background leakage rates were suggested to be at most 0.25, 0.25 and 1.5 L/connection/h for residential plumbing, service pipes between curbstops and customer meters, and service pipe connections (including curbstop valve, pipe between main and curbstop and service tap), respectively, and at most 20 L/km/h for distribution mains. Losses of systems in average and poor conditions would be twice and at least 3 times losses of systems in good condition. It's not clear what were the bases used in Report E (1994) for establishing this relationship as well as for revising measured average values by +25 to +100%.

Similarly, the basis for using an exponent equal to 1.5 for the power relationship between background leakage and pressure by Lambert (2001) is not clear. Supporting data for this exponent is not well document in the literature. Interestingly, a linear relationship is used between pressure and Unavoidable Real Annual Loss (UARL), which includes losses from background leaks and losses from reported and unreported leaks for a system that is in good condition and undergoing intensive active leakage monitoring and repair (Lambert et al., 1999) and (Alegre et al., 2000). This seems to follow the practice of using a power exponent equal to 1 for systems before undergoing leak detection and repair, which is based the average exponent obtained from numerous tests on actual distribution systems in Japan, U.K. and Brazil (Lambert, 2001). Background leakage in these systems was most likely not the major component of total leakage. However, it is the major component in UARL, constituting approximately two-thirds of total loss.

Consequently, it was initially believed that the use of N_1 = 1 for UARL may not be representative. Since current practice is to use N_1 = 1.5 for background leakage and 0.5 for reported and unreported recoverable leaks, N_1 for UARL was expected to be greater than 1. UARL obtained using the equation proposed by Lambert et al. (1999) with a combined N_1 = 1 and UARL obtained using N_1 =1.5 and 0.5 for background leakage and reported/unreported leakage components, respectively, are presented in Table 7.

As can be seen in Figure 40, N_1 was 1.032 for the best-fit power relationship of UARL values obtained using N1 =1.5 and 0.5 for the background leakage and reported/unreported leakage components, respectively. Subsequently, the current assumption of a linear pressure-UARL relationship is apparently representative. However, the error in the Infrastructure Leakage Index (ILI) is large, as seen in Table 7 and Figure 41. In the pressure range from 5 to 150 psi, the ILI error range is –7.45 to +34.55%.

Also presented in Table 7 are UARL values obtained using N_1 =1.5 and 1 for background leakage and reported/unreported recoverable leakage components, respectively. As can be seen in Figure 42, N_1 was 1.305 for the best-fit power relationship of these UARL values. Subsequently, the current assumption of a linear pressure-UARL relationship may be unreasonable and also as expected the error in the Infrastructure Leakage Index (ILI) is large, as can be seen in Table 7 and Figure 41. In the pressure range from 5 to 150 psi, the ILI error range is –50.53 to 32%.

A problem that has recently surfaced regarding ILI was its suitability as a performance indicator for pressure management. Although common wisdom suggests that pressure management can be expected to improve the leakage management performance of utilities, Preston and Sturm (2007) suggested that pressure management has no effect on ILI. This is based on the previously noted practice that for actual systems, leakage almost varies linearly with pressure (Lambert, 2001). Actually, it may be argued that if N_1 = 1.5 is strictly used for background leakage and N_1 = 0.5 or 1 is used for reported and unreported leaks in both UARL and CARL (current annual real losses), the ILI of networks comprised mainly of rigid pipes may increase with pressure management. This can be seen in Figure 43 for a system whose unreported/reported leakage is 10 times that assumed in the corresponding component of UARL, and having connection density of 80 connections/km and average length of 15 metres for service pipes from curbstops to customer meters.

Finally, UARL used in current practice may not be applicable for Canadian and other northern water pipe networks. This is because current assumptions regarding frequencies, durations and flow rates of reported and unreported detectable leaks in various components of pipe networks may not be representative. For example, leak frequency currently used for service pipes is more than twice that of distribution mains. However, based on informal probing, the majority of leaks in Canadian pipe networks appear to be in distribution mains.

The break frequency of distribution mains depends to a large degree on pipe type. A survey of 21 Canadian water pipe networks (Rajani and McDonald, 1995) revealed that average break frequency of cast iron mains in 1992/1993 was about 0.36 break/km/yr, almost 4 times the break frequency for ductile iron pipes. Overall average break frequency was about 0.2 break/km/yr, whereas the break frequency currently used for UARL is 0.13 breaks/km/yr. Further, it's currently assumed that 1 break of every 20 in distribution pipes is unreported. For pipes in Canada, it's expected that a higher proportion of breaks is unreported, i.e., do not surface, because pipes are buried significantly deeper than in the U.K. and Germany (where data for current practice originated).

8 SUGGESTED ESTIMATES AND PROCEDURES

Background Leakage

In the absence of reliable information about DMA-specific background leakage rate, it is proposed to estimate it using the following equation for distribution networks in good condition:

Eq. (8): $\text{Night background leakage} = \left(A \times L_m + B \times N_c + C \div 15 \times L_p \times N_c\right) \times \left(\frac{P}{71}\right)^{N_1}$, in L/h

where P is average pipe pressure in psi; N_1 is equal to 0.55, 1.11, 1.35 and 1.5 for asbestos cement, ductile iron, cast iron and PVC pipes respectively; L_m is total length of distribution pipes in km; N_c is total number of service connections; L_p is average length in metres of service connection pipes between curb stops and customer water meters; and A, B and C are constants equal to 24 L/km/h, 1.5 L/connection/h and 0.4 L/connection/hour corresponding to rates of leakage components at 71 psi (50 metres) pressure in distribution mains, service connection pipes from mains to curbstops and 15-metre long service connection pipes after curbstops, respectively. N_1 for PVC pipes was not measured in this study but is based on current practice (Lambert, 2001). N_1 for asbestos cement pipes was based on data collected in this study in Regina.

To convert the rate in Eq. (8) from L/h to L/day, multiply by 24 hours or by an appropriate hour-to-day conversion factor if diurnal pressure fluctuates significantly.

Residential Night Demand

In the absence of reliable information about DMA-specific average residential demand, it is proposed to that average residential night demand be taken equal to 3 and 2 L/household/h for older and newer areas, respectively, over the period from 1:30 to 3:30 or 2:00 to 3:00 AM.

Recoverable Leakage

<u>Temporary DMAs (No AMR)</u>

The following procedure is proposed to estimate recoverable leakage in temporary DMAs between 11:00 PM and 6:00 AM for areas that do not have residential water meters on an AMR system:

1. Establish the boundary of the test area so that it includes 500 to 2000 service connections. Have as much natural boundaries as possible to minimize the number of boundary valves.
2. Connect a temporary water supply rig across the boundary of the DMA during any night of the week (excluding Friday and Saturday and the lawn-watering season). The rig should include a properly sized inlet flow meter, data logger, PRV on a bypass (if available), pressure gauge, and at least one inline shutoff valve (if only one valve is used, it should be placed after the flow meter). Open the Water supply.
3. Install a pressure gauge and logger at the fire hydrant nearest to the point of average pressure in the DMA.
4. Isolate the DMA by closing all boundary valves, as early in the night as possible to allow time for proving the DMA. The earliest time is dictated by the ability of the temporary water supply to meet demand and pressure requirements.
5. Prove the boundary of the DMA as follows: (a) reduce average pressure inside the DMA by ~25 psi via an inlet PRV or by throttling an inlet valve, (b) sound all boundary valves using an aquaphone or an electronic listening device, (iii) exercise noisy valves and adjust by tightening or if noise persists by back turning slightly, and (iv) adjust the boundary of the DMA to exclude passing valves that cannot be fixed.
6. Start logging of water flow at the DMA's inlet and pressure at the average location as early in the night as possible. Suggested data logging interval is 1 minute.

7. Determine average DMA supply flow rate over 1:30-3:30 or 2:00-3:00 AM.
8. Repeat steps 2 to 7 over a statistically appropriate number of nights (excluding Friday and Saturday, and outside the lawn-watering season).
9. Determine overall average DMA supply flow rate from all nights.
10. Estimate background leakage rate using Eq. (8).
11. Estimate average rate of residential night demand over 1:30-3:30 or 2:00-3:00 AM using 3 and 2 L/household/h for older and newer areas, respectively.
12. Estimate recoverable leakage as supply flow rate minus rate of background leakage minus rate of residential night demand.
13. Convert rate of recoverable leakage from L/h to L/day by multiplying by 24 hours or by an appropriate hour-to-day conversion factor if diurnal pressure fluctuates significantly.
14. If recoverable leakage rate exceeds the economic leakage rate for the area:

    Intervene (i.e., leak survey and repair).

    Else,

    Go to step 17.
15. Repeat steps 1 to 13 to assess effectiveness of leak survey and repair.
16. Undertake a toilet leakage awareness campaign in the DMA (distribution of information leaflets and free detection tablets, etc.) if the cost of the campaign is less than the accumulated cost of lost water estimated theoretically as recoverable leakage rate multiplied by half the time period since the last awareness campaign was undertaken, assuming leakage rate rises linearly and is mostly due to visually and audibly undetectable toilet leaks.
17. End

If the sample of measured DMA supply flow rates comes from a “normal” population, it can be ascertained with a probability of $1-\alpha$ that the true mean value, μ, of the supply flow rate falls in the following range:

Eq. (9): $$\bar{x} - \frac{t_{\alpha/2} \cdot s}{\sqrt{n}} \le \mu \le \bar{x} + \frac{t_{\alpha/2} \cdot s}{\sqrt{n}}$$

where *n* is the size of the sample, i.e., number of nights over which flow rate is measured, $\bar{x}$ is the mean of the sample, *s* is the standard deviation of the sample and $t_{\alpha/2}$ is the value of a variable (equal to $\frac{\bar{x} - \mu}{s/\sqrt{n}}$, which has a student-t distribution with *n*-1 degrees of freedom) for which the area to its right under the *t*-distribution is equal to α. For α equal to 0.05 and *n* equal to 3,5,10 and 30, $t_{\alpha/2}$ is equal to 4.303, 2.776,2.262 and 2.045, respectively.

The economic leakage rate is derived in Appendix F. For permanent DMAs, it is exceeded when the accumulated cost of lost water exceeds the cost of intervention. For temporary DMA, the following economic leakage rate may be used:

Eq. (10): $$ELR = RL_{DMA}F_oT_I^{optimum}$$

where *R* is the average volume of water lost in m^3 per year per leak, L_{DMA} is the length of distribution pipes in the DMA in km, F_o is the frequency of unreported leaks per km of pipe per year, and $T_I^{optimum}$ is the theoretically optimum intervention period given by:

Eq. (11): $$T_I^{optimum} = \sqrt{\frac{2C_{survey}^{DMA}}{cRL_{DMA}F_o}}$$

where *c* is the marginal cost of lost water (\$/m^3), and C_{survey}^{DMA} is the cost of acoustically surveying the whole DMA.

Temporary DMAs (with AMR)

The following procedure is proposed to estimate recoverable leakage in temporary DMAs between 11:00 PM and 6:00 AM for areas that have residential water meters on an AMR system (assuming it is a drive-by type, i.e. meter readings are not simultaneous):

1. Establish the boundary of the test area so that it includes 500 to 2000 service connections. Have as much natural boundaries as possible to minimize the number of boundary valves.
2. Connect a temporary water supply rig across the boundary of the DMA during any night of the week (including Friday and Saturday and the lawn-watering season). The rig should include a properly sized inlet flow meter, data logger, PRV on a bypass (if available), pressure gauge, and at least one inline shutoff valve (if only one valve is used, it should be placed after the flow meter). Open the Water supply.
3. Install a pressure gauge and logger at the fire hydrant nearest to the point of average pressure in the DMA.
4. Isolate the DMA by closing all boundary valves, as early in the night as possible. The earliest time is dictated by the ability of the temporary water supply to meet demand and pressure requirements.
5. Prove the boundary of the DMA as follows: (a) reduce average pressure inside the DMA by ~25 psi via an inlet PRV or by throttling an inlet valve, (b) sound all boundary valves using an aquaphone or an electronic listening device, (iii) exercise noisy valves and adjust by tightening or if noise persists by back turning slightly, and (iv) adjust the boundary of the DMA to exclude passing valves that cannot be fixed.
6. Start logging of water flow at the DMA's inlet and pressure at the average location as early in the night as possible. Suggested data logging interval is 1-minute.
7. Collect AMR readings of all residential and non-residential water meters in the DMA twice: (a) at the same start time (or slightly later) of step 6, and

(b) after 3 to 4 hours. The longer the period, the less significant the effect of time lag between AMR readings and the inlet meter.

8. Terminate logging of water flow at the DMA's inlet and pressure at the average location soon after collecting the second set of AMR readings in step 7.
9. Determine the following for the water balance period:
   a. Start and end times by averaging the times of readings in the first and second AMR sets, respectively.
   b. Average of pressure logged at the average location.
   c. Volume in litres of background leakage based on Eq. (8) and pressure determined in step 9 (b).
   d. Volume in litres of water demand based on AMR readings.
   e. Volume in litres of water supply based on flow log at the DMA's inlet.
10. Estimate recoverable leakage rate as volume of water supply minus volume of background leakage minus volume of water demand divided by the duration of water balance period in hours.
11. Convert the rate of recoverable leakage from L/h to L/day by multiplying by 24 hours or by an appropriate hour-to-day conversion factor if diurnal pressure fluctuates significantly.
12. If recoverable leakage rate exceeds the economic leakage rate for the DMA:

    Intervene (i.e., leak survey and repair).

    Else,

    Go to step 14.
13. Repeat steps 2 to 11 to assess effectiveness of leak survey and repair.
14. End

<u>Permanent DMAs (No AMR)</u>

The following procedure is suggested to estimate recoverable leakage for permanently set up DMAs that do not have customer water meters on an AMR system:

1. Establish the boundary of the test area so that it includes 500 to 2000 service connections. Have as much natural boundaries as possible to minimize the number of boundary valves.
2. Install and start a properly sized inlet flow meter, pressure sensor, PRV, appropriate inline shutoff valves and a 2-channel data logger (preferably of the wireless type). Suggested data logging interval is 1-minute.
3. Close all boundary valves.
4. Prove the boundary of the DMA as follows: (a) reduce average pressure inside the DMA by ~25 psi via the inlet PRV (or by throttling an inlet valve), (b) sound all boundary valves using an aquaphone or an electronic listening device, (iii) exercise noisy valves and adjust by tightening or back turning slightly, and (iv) replace valves that cannot be fixed or adjust the boundary of the DMA to exclude them.
5. Install a temporary pressure gauge and logger at the fire hydrant nearest to the point of average pressure in the DMA. Log pressure over a 24-h period. Determine a pressure correction factor as the ratio between average pressures at the average location and the DMA's inlet.
6. When the DMA is first established, conduct a detailed leak survey and repair all leaks. The survey must be conducted by sounding all valves and curb stops; alternatively, a correlator-based survey may suffice.
7. Estimate residential demand rate over the period 1:30-3:30 AM or 2:00-3:00 AM as number of households times 3 and 2 L/household/h for older and newer areas, respectively.
8. Immediately following step 6, estimate average leakage rate during the lawn-watering season. The average rate should be determined over a statistically appropriate number of nights, excluding Friday and Saturday nights. Leakage rate is to be estimated as supply flow rate over the period 1:30-3:30 AM or 2:00-3:00 AM minus estimated residential night demand. Background leakage determined as such includes distribution pipe losses; residential plumbing losses; and water drawn by lawn sprinklers.

9. Estimate background leakage rate outside the lawn-watering season using Eq. (8).
10. Estimate average recoverable leakage rate over the period 1:30-3:30 or 2:00-3:00 AM as supply flow rate measured over a statistically appropriate number of nights minus background leakage rate corresponding to appropriate season minus estimated residential night demand.
11. Convert the rate of recoverable leakage from L/h to L/day by multiplying by 24 hours or by an appropriate hour-to-day conversion factor if diurnal pressure fluctuates significantly.
12. If recoverable leakage exceeds the economic rate (i.e., the accumulated cost of lost water exceeds the cost of intervention) then:

    Intervene (i.e., leak survey and repair).

    Else,

    Go to step 10
13. If <u>outside the lawn watering season</u>, undertake a toilet leakage awareness campaign in the DMA if the cost of the campaign is less than the accumulated cost of lost water estimated theoretically as the recoverable leakage rate multiplied by half the time period since the last awareness campaign was undertaken (assuming the toilet leakage rate rises linearly and is mostly due to visually and audibly undetectable toilet leaks).
14. Go to Step 10.

<u>Permanent DMAs (with AMR)</u>

The following procedure is suggested to estimate recoverable leakage for permanently set up DMAs that have customer water meters on an AMR system (assuming it is drive-by type, i.e., meter readings are not simulataneous):

1. Establish the boundary of the test area so that it includes 500 to 2000 service connections. Have as much natural boundaries as possible to minimize the number of boundary valves.

2. Install and start a properly sized inlet flow meter, pressure sensor, PRV, appropriate inline shutoff valves and a 2-channel data logger (preferably of the wireless type). Suggested data logging interval is 1-minute.
3. Close all boundary valves.
4. Prove the boundary of the DMA as follows: (a) reduce average pressure inside the DMA by ~25 psi via the inlet PRV (or by throttling an inlet valve), (b) sound all boundary valves using an aquaphone or an electronic listening device, (iii) exercise noisy valves and adjust by tightening or back turning slightly, and (iv) replace valves that cannot be fixed or adjust the boundary of the DMA to exclude them.
5. Install a temporary pressure gauge and logger at the fire hydrant nearest to the point of average pressure in the DMA. Log pressure over a 24-h period. Determine a pressure correction factor as the ratio between average pressures at the average location and the DMA's inlet.
6. Collect AMR readings of all residential and non-residential water meters in the DMA twice, at the beginning and end of a 24-h water balance period starting at any time between 12:00 AM and 4:00 AM.
7. Determine the following for the water balance period:
    a. Start and end times by averaging the times of readings in the first and second AMR sets, respectively.
    b. Average pressure in the DMA as average inlet pressure times the pressure correction factor determined in step 5.
    c. Volume in litres of background leakage based on Eq. (8) and average DMA pressure determined in step 7 (b).
    d. Volume in litres of water demand based on AMR readings.
    e. Volume in litres of water supply based on flow log at the DMA's inlet.
8. Estimate recoverable leakage rate as volume of water supply minus volume of background leakage minus volume of water demand divided by the duration of water balance period in hours.

9. Convert the rate of recoverable leakage from L/h to L/day, multiply by 24 hours (diurnal pressure fluctuation is implicitly accounted for by the use of 24-h average pressure in step 7 (c)).
10. If recoverable leakage exceeds the economic rate (i.e., the accumulated cost of lost water exceeds the cost of intervention) then:
    Intervene (i.e., leak survey and repair).
    Repeat steps 6 to 9 to assess effectiveness of leak survey and repair.
11. If the AMR system is of the fixed-network type Go to step 6 on the following day, or if AMR is of the drive-by type Go to step 6 after a suitably long period, that's not longer than $T_I^{optimum}$ given by Eq. (11).

For high-resolution AMR systems (e.g., those capable of collecting meter readings with an accuracy of 1 litre or less), a water balance over few night hours may be accurate enough (e.g., 12:00 AM to 4:00 AM). If the AMR system does not have high resolution, water balance should be carried out over at least a 24-h period, with AMR of flow meters collected during 1:00-5:00 AM for systems incapable of synchronous reading of all meters in the DMA. The water balance period should be carried out over multiples of whole days (i.e., 24, 48, 72… hours).

9 CONCLUSIONS

The objective of this study was to develop enhanced leakage management methods for municipal water pipe networks in Canada. Emphases were on the district metered-area method and included determination of night residential demand; development of an empirical model for estimation of background leakage rates and/or development of analytical procedure(s) that can derive it directly; derivation of relationships between leakage rate and pipe pressure; and comparison of the performance of different acoustic leak detection methods.

The study involved extensive measurement and analysis of flow and pressure nightlines under controlled conditions at 2 residential DMAs in Ottawa (one comprised of ductile iron pipes and the other mainly of cast iron pipes) and at 2 residential DMAs in Regina (one comprised of asbestos-cement pipes and the other of PVC pipes.) Planned fieldwork in Halifax could not be performed due to administrative difficulties at NRC.

Based on field measurements and analysis of night flow and pressure in the ductile and cast iron pipe DMAs in Ottawa, the following conclusions were made for residential night demand; background leakage rates, recoverable leakage and relationships between leakage rate and pipe pressure (results of fieldwork in Regina are presented in report 1):

1. Background leakage rates were ~3, 2.26 and 1.65 L/connection/h at ~92, 71.5 and 53 psi pipe pressure, respectively, for ductile iron pipes in Orleans. The best-fit power relationship between background leakage and pressure was $L_{Background} = 0.0203 \times P^{1.11}$, where leakage is in L/connection/h and pressure is in psi. The N_1 exponent of this power relationship is significantly lower than the exponent of 1.5 used in current practice.

2. For cast iron pipes in Meadowlands, there was significant variation in background leakage rate from night to night and sometimes over the same night. This variation did not correspond to pressure stepping; it was attributed to variable plumbing losses, especially toilet leakage in this area with many old homes. On few nights, background leakage rates were ~3 and 1.5 L/connection/h at ~85 and 53 psi pipe pressure, respectively. These rates are not believed to include significant toilet leakage because they are reasonably close to rates predicted based on U.K. data ascertained to be toilet leakage free. The best-fit power relationship between background leakage and pressure was $L_{Background} = 0.0075 \times P^{1.351}$, where leakage is in L/connection/h and pressure is in psi. The N_1 exponent of this power relationship is relatively close to the exponent of 1.5 used in current practice.

3. The best-fit power relationship between the highest toilet leakage and pressure in the Meadowlands area was $L_{Toilet} = 0.0103 \times P^{1.43}$, where leakage is in L/connection/h and pressure is in psi. Toilet leakage was estimated as the highest measured minimum night flow rate minus background leakage estimated using the power relationship $L_{Background} = 0.0075 \times P^{1.35}$. The highest toilet leakage given by the preceding relationship is likely underestimated since night flow was not monitored over many nights and subsequently the highest possible minimum night flow rate may have been missed. The power exponent for toilet leakage varied from 1.06 to 1.43; average exponent was ~1.2.

4. Background leakage rates predicted by current practice with estimates at a reference pressure of 71 psi (50 metres) and N_1 exponents obtained in this study were reasonably close to rates based on the best-fit power relationship of measured rates. A better overall agreement, taking into account flow measurements in both Regina and Ottawa, was obtained by slightly adjusting constants in the equation used in current practice as follows:

   $$\text{Night background leakage} = \left(A \times L_m + B \times N_c + C \div 15 \times L_p \times N_c\right) \times \left(\frac{P}{71}\right)^{N_1}, \text{ in L/h}$$

   where P is average pipe pressure in psi; N_1 is equal to 0.55, 1.11, 1.35 and 1.5 for asbestos cement, ductile iron, cast iron and PVC pipes respectively; L_m is total length of distribution pipes in km; N_c is total number of service connections; L_p is average length in metres of service connection pipes between curb stops and customer water meters; and A, B and C are constants equal to 24 L/km/h, 1.5 L/connection/h and 0.4 L/connection/hour corresponding to rates of leakage components at 71 psi (50 metres) pressure in distribution mains, service connection pipes from mains to curbstops and 15-metre long service connection pipes after curbstops, respectively. N_1 for PVC pipes was not measured in this study but was based on current practice (Lambert, 2001).

5. Difficulties were encountered in proving the boundaries of test areas used to measure background leakage rates and the N_1 exponent in the older Meadowlands area. It was likely that some boundary valves were passing but this could not be detected using acoustic listening equipment while the valves were subjected to a moderate pressure differential. It may be that the flow rate of passing water was too low, i.e., flow was laminar, to create detectable noise. Or it may be that the gates of some valves were worn out and moved somehow after the valves were sounded. This could happen due to the removal of valve keys that rested on top of valve nuts during sounding or due to the change of differential pressure across valve gates after sounding, during pressure stepping. Subsequently, the accuracy of background leakage rates based on minimum night flows may not be possible to ascertain for old pipe areas unless a zero-pressure test is conducted to prove the area's boundary. Most pipe network operators do not permit zero-pressure tests.

6. It is recommended that N_1 tests for background leakage be conducted in as large areas as possible with very few boundary valves (ideally none other than the inlet valve). The test area should not include active major nonresidential users and no more than ~400 households. Boundary valves should be sounded using an electronic listening device at the highest possible differential pressure.

7. Residential night demand was generally higher than the currently used estimate of 1.7 L/household/h. This was especially the case during the lawn-watering season, with demand being up to 5 times the currently used estimate. Consequently, leakage rates based on analysis of minimum night flows may be in error by up to 200 litres per minute for a typical DMA of 2000 connections. This is equivalent to 8 service pipe leaks or a distribution pipe break. Demand varied significantly from night to night. Consequently, night flow analysis during the lawn-watering season will lead to inaccurate results if residential demand is not appropriately

estimated, e.g., using automatic meter reading (AMR) of all residential and nonresidential water meters in the DMA.

8. Outside the lawn-watering season, residential night demand varied only slightly from night to night. However, it generally decreased with time over the same night.

9. Overall average residential night demand over the period 1:30-3:30 AM outside the lawn-watering season in the older Meadowlands area was higher by about 1 L/household/h than demand in the newer Orleans area. Overall average for the 1:30-3:30 AM and 2:00-3:00 AM periods was equal to 2.08 and 2.03 L/household/h, respectively, in the Orleans area and 3 and 3.2 L/household/h, respectively, in the Meadowlands area. The difference was attributed to a larger proportion of newer homes in the Orleans area that may have water-conserving toilets, e.g., having small tanks or a dual flush feature.

10. It may be necessary to use DMA-specific residential night demand in order to accurately determine the rate of recoverable leakage based on analysis of DMA night flow. In the absence of this information, it is proposed that analysis of night flow be based on DMA supply flow rate averaged over the period 1:30-3:30 AM or 2:00-3:00 AM (instead of using the minimum of a 60-minute moving average) and assuming an average residential demand of 3 and 2 L/household/h for older and newer areas, respectively (instead of the currently used estimate of 1.7 L/household/h).

11. There can be significant spurious recoverable leakage determined from the analysis of night flows during the lawn-watering season. Also, there can be significant night-to-night variation in the rate of recoverable leakage. In one instance recoverable leakage rate increased with decreasing pipe pressure. Significant spurious leakage and its variation are a result of night use of lawn water sprinklers. Results obtained during this season should be treated with caution.

12. Outside the lawn-watering season, there can be relatively high spurious recoverable leakage (equal to or greater than ~70 and 100 litres per minute in the Orleans and Meadowlands DMAs, respectively), although no leaks could be detected by acoustic surveys. This can be due to ineffective leak surveys, underestimation of residential demand, and (or) plumbing leakage in residences and major establishments such as schools. In the Orleans ductile iron pipe DMA, recoverable leakage increased slightly when pressure was reduced from ~88 to 55 psi, contrary to expectation. Also, in the Meadowlands cast iron DMA, recoverable leakage had in one instance almost the same rate at significantly different pressures, while at the same pressure it varied by ~30 litres per minute from night to night. Leakage-pressure power relationship exponent N_1 varied between 0 and 3, which may be taken as indication of the large variation of mainly toilet leakage from night to night. Residential demand measured in small areas in the DMAs did not vary significantly from night to night.

13. There was only a small difference between recoverable leakage rates obtained using flow rates averaged over 1:30-3:30 AM, 2:00-3:00 AM, 2:00-4:00 AM, and 2:30-3:30 AM. However, as expected, leakage rate obtained using minimum of 60-minute moving average flow rate was always lower than the rate obtained using average flow rates over the above fixed periods.

14. Detailed procedures were presented for setting up either temporary or permanent DMAs to estimate recoverable leakage rate, with or without AMR.

Poor accuracy was obtained for night flow components calculated analytically using a system of linear algebraic equations formulated with currently used or measured N_1 exponents and assuming constant residential night demand. Most likely, this is due to the assumption of constant residential night demand and to errors in N_1 exponents. The system of linear equations utilized to analytically identify components of night flow is very sensitive to very small variation in residential demand and to a lesser degree to N_1 exponents.

Fieldwork in Ottawa to compare different leak detection strategies could not be performed because the city as a result of stretched resources and unexpected heavy workload could not provide the required support. Instead, description of different acoustic leak detection strategies, discussion of their pros and cons, and reported experiences with their performance were presented. In addition, interesting results emerged from acoustic listening and correlation surveys that were undertaken to determine the source of the high leakage in the cast iron and asbestos-cement pipe DMAs in Ottawa and Regina, respectively.

Hydrant listening and correlation-based surveys by city staff in Ottawa did not succeed in locating a large ~600 L/minute leak caused by a full circumferential break in a 200 mm ∅ cast iron pipe. Failure to detect such leaks may be due to faulty equipment or due to significant leak noise attenuation caused by frozen soil in winter or by short plastic pipe sections or splices used for pipe repair or as leads when old fire hydrants are replaced. These difficulties should be factored in acoustic survey strategies. Leak detection equipment should be verified frequently. Also, consideration should be given to the use of hydrophones instead of vibration sensors at fire hydrants or to probably attaching acoustic sensors at inline valves.

Hydrant listening surveys by city staff in Regina also did not succeed in detecting the source of leakage estimated at 393 L/minute in the asbestos-pipe DMA, which was later found to include a large leak from a partial circumferential crack in a 152 mm ∅ AC pipe. Noise from this large leak could not even be heard using an electronic listening device at service connection curbstops that were only few meters away. Failure of listening surveys can be attributed to the poor noise propagation characteristics of AC pipes, as well as the widespread use of short plastic pipe sections or splices for AC pipe repair or as leads when old fire hydrants are replaced. The same leak was also initially missed by a correlater-based leak survey with hydrophone sensors due to out-of-bracket interference by noise from the regeneration of water softeners, which are commonly used in Regina.

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Table 1: Average residential demand (in L/household/h) based on supply flow measurements in Orleans ductile iron sub-DMA

	A: Minimum flow rate equal to background leakage rate including residential plumbing losses[1]						B: Average supply to sub-DMA						C: Average residential demand excluding plumbing losses (B minus A)						D: Overall average residential demand
Period	26 June 2006	27 June 2006	28 June 2006	17 Oct 2006	18 Oct 2006	19 Oct 2006	26 June 2006	27 June 2006	28 June 2006	17 Oct 2006	18 Oct 2006	19 Oct 2006	26 June 2006	27 June 2006	28 June 2006	17 Oct 2006	18 Oct 2006	19 Oct 2006	17,18,19 Oct 2006
1:00 AM to 4:30 AM	3.07	3.07	3.05	2.98	1.60	2.25	6.38	5.76	8.10	5.15	4.04	4.43	3.31	2.69	5.20	2.17	2.44	2.18	2.26
1:00 AM to 3:00 AM	3.06	3.10	3.07	3.00	1.60	2.25	6.65	6.10	8.05	5.59	4.45	4.91	3.59	3.00	5.15	2.60	2.85	2.66	2.70
1:30 AM to 3:30 AM	3.07	3.09	3.05	3.01	1.61	2.25	6.32	5.55	7.32	4.98	3.79	4.35	3.25	2.46	4.42	1.98	2.18	2.09	2.08
2:00 AM to 4:00 AM	3.07	3.07	3.03	2.99	1.61	2.26	6.09	5.36	7.50	4.84	3.88	3.68	3.02	2.29	4.60	1.85	2.27	1.42	1.85
1:00 AM to 2:00 AM	3.05	3.09	3.09	2.98	1.60	2.24	7.28	6.69	8.90	6.08	4.66	6.17	4.23	3.60	6.00	3.10	3.06	3.93	3.36
1:30 AM to 2:30 AM	3.06	3.10	3.08	3.00	1.61	2.25	6.55	5.90	8.71	4.96	3.35	4.87	3.49	2.81	5.81	1.96	1.74	2.62	2.11
2:00 AM to 3:00 AM	3.07	3.11	3.04	3.01	1.60	2.26	6.04	5.55	7.23	5.10	4.23	3.64	2.97	2.45	4.33	2.09	2.63	1.38	2.03
2:30 AM to 3:30 AM	3.07	3.08	3.03	3.01	1.60	2.26	6.08	5.22	5.90	5.00	4.22	3.83	3.01	2.14	3.00	1.99	2.61	1.57	2.06
3:00 AM to 4:00 AM	3.07	3.03	3.03	2.98	1.61	2.26	6.12	5.16	7.78	4.58	3.53	3.72	3.05	2.12	4.88	1.60	1.92	1.46	1.66

1 Background leakage is based on relationship in Figure 33 using average pressure in the corresponding period. Overall average pressure in the sub-DMA was ~92.5 psi on 26/27/28 June, 92 psi on 17 Oct, 53 psi on 18 Oct, and 71.5 on 19 Oct.

Table 2: Average residential demand (in L/household/h) based on supply flow measurements in Meadowlands cast iron sub-DMA (combined purple & blue zones)

	A			B			C			D
	Minimum flow rate equal to background leakage rate including residential plumbing losses[1]			Average supply to sub-DMA			Average residential demand excluding plumbing losses (B minus A)			Overall average residential demand
Period	19 April 2007	23 April 2007	24 April 2007	19 April 2007	23 April 2007	24 April 2007	19 April 2007	23 April 2007	24 April 2007	19,23,24 April 2007
1:00 AM to 4:30 AM	2.91	1.42	3.49	5.71	4.46	6.83	2.79	3.04	3.33	3.05
1:00 AM to 3:00 AM	2.88	1.42	3.49	6.13	5.07	7.32	3.25	3.65	3.83	3.58
1:30 AM to 3:30 AM	2.91	1.42	3.49	5.78	4.52	6.49	2.86	3.11	3.00	2.99
2:00 AM to 4:00 AM	2.94	1.42	3.49	5.43	4.03	6.36	2.49	2.61	2.87	2.66
1:00 AM to 2:00 AM	2.86	1.42	3.49	6.32	5.53	7.79	3.46	4.11	4.30	3.96
1:30 AM to 2:30 AM	2.88	1.42	3.49	6.17	4.79	6.73	3.29	3.37	3.23	3.30
2:00 AM to 3:00 AM	2.90	1.41	3.49	5.95	4.61	6.85	3.05	3.20	3.36	3.20
2:30 AM to 3:30 AM	2.95	1.42	3.49	5.39	4.26	6.26	2.44	2.84	2.76	2.68
3:00 AM to 4:00 AM	2.98	1.43	3.49	4.90	3.44	5.86	1.92	2.02	2.37	2.10

1 Background leakage is based on relationship in Figure 35 using average pressure in the corresponding period, except for the night of 24 April for which actual measured level is used. Overall average pressure in the sub-DMA was 84, 53 and 65 psi on 19, 23 and 24 April 2007, respectively.

Table 3: Recoverable leakage rate in the ductile iron pipe DMA in Orleans

Night	Period	Average pressure (psi)	Total flow (L/minute)	Background leakage (L/minute)[1]	Residential demand (L/minute)	Non-residential demand (L/minute)	Recoverable leakage (L/minute)
19 June 2006	1:30-3:30	88.4	253.2	88.3	61.1	20	83.8
	2:00-3:00	88.7	247.8	88.7	61.1	20	78.0
	2:00-4:00	88.9	246.0	88.9	55.0	20	82.1
	2:30-3:30	89.1	243.1	89.1	55.0	20	79.0
	Minimum 60-minute average	89.5	242.0	89.5	71.5	20	60.9
20 June 2006	1:30-3:30	48.4	243.2	45.4	61.1	20	116.6
	2:00-3:00	48.5	232.0	45.4	61.1	20	105.4
	2:00-4:00	48.4	254.4	45.3	55.0	20	134.1
	2:30-3:30	48.4	257.3	45.3	55.0	20	137.0
	Minimum 60-minute average	48.4	225.0	45.4	71.5	20	88.1
21 June 2006	1:30-3:30	60.8	288.8	58.4	61.1	20	149.3
	2:00-3:00	60.7	296.2	58.3	61.1	20	156.8
	2:00-4:00	60.6	312.6	58.2	55.0	20	179.4
	2:30-3:30	60.7	302.5	58.3	55.0	20	169.2
	Minimum 60-minute average	60.6	263.0	58.2	71.5	20	113.3
22 June 2006	1:30-3:30	71.2	376.3	69.6	61.1	20	225.6
	2:00-3:00	72.6	321.6	71.0	61.1	20	169.5
	2:00-4:00	69.1	445.1	67.2	55.0	20	302.8
	2:30-3:30	68.7	427.3	66.9	55.0	20	285.4
	Minimum 60-minute average	72.0	317.0	70.4	71.5	20	155.1
24 October 2006	1:30-3:30	88.2	242.9	88.1	61.1	20	73.7
	2:00-3:00	89.0	242.9	89.0	61.1	20	72.7
	2:00-4:00	87.9	230.8	87.7	55.0	20	68.1
	2:30-3:30	88.1	221.6	88.0	55.0	20	58.5
	Minimum 60-minute average	88.0	219.0	87.9	71.5	20	39.6
26 October 2006	1:30-3:30	55.2	208.4	52.5	61.1	20	74.7
	2:00-3:00	55.3	212.1	52.5	61.1	20	78.4
	2:00-4:00	55.2	209.1	52.5	55.0	20	81.6
	2:30-3:30	55.2	206.3	52.5	55.0	20	78.8
	Minimum 60-minute average	55.0	204.0	52.3	71.5	20	60.2

1 Background leakage is based on relationship in Figure 33 using average pressure in the corresponding period.

Table 4: Recoverable leakage rate in the cast iron pipe DMA in Meadowlands

Night	Period	Average pressure (psi)	Total flow (L/minute)	Background leakage (L/minute)[1]	Toilet leakage (L/minute)[2]	Residential demand (L/minute)	Total recoverable leakage (L/minute)	Recoverable excluding toilet leakage (L/minute)
16 April 2007	1:30-3:30	81.03	242.68	43.1	124.4	67.8	131.8	7.4
	2:00-3:00	81.11	242.91	43.1	124.6	67.8	132.0	7.4
	2:00-4:00	81.5	233.2	43.4	125.4	61.0	128.8	3.3
	2:30-3:30	81.6	231.9	43.5	125.7	61.0	127.4	1.7
	60-minute avg	81.0	217.0	43.0	124.4	52.9	121.1	-3.3
29 April 2007	1:30-3:30	80.33	270.47	42.6	122.9	67.8	160.1	37.2
	2:00-3:00	80.38	268.96	42.6	123.0	67.8	158.6	35.5
	2:00-4:00	80.8	261.2	42.9	123.9	61.0	157.3	33.4
	2:30-3:30	80.7	264.6	42.8	123.8	61.0	160.7	37.0
	Minimum 60-minute average	80.8	233.3	42.9	123.9	52.9	137.5	13.6
30 April 2007	1:30-3:30	46.49	176.86	20.3	56.3	67.8	88.7	32.5
	2:00-3:00	46.76	177.93	20.5	56.7	67.8	89.6	32.9
	2:00-4:00	46.5	173.8	20.4	56.3	61.0	92.4	36.1
	2:30-3:30	46.9	173.8	20.5	56.9	61.0	92.2	35.4
	Minimum 60-minute average	46.5	165.4	20.3	56.3	52.9	92.2	35.9
1 May 2007	1:30-3:30	64.23	203.79	31.5	89.3	67.8	104.5	15.3
	2:00-3:00	64.41	207.02	31.6	89.6	67.8	107.6	18.0
	2:00-4:00	63.8	192.3	31.2	88.4	61.0	100.1	11.6
	2:30-3:30	63.8	191.5	31.2	88.4	61.0	99.3	10.9
	Minimum 60-minute average	64.0	173.3	31.3	88.8	52.9	89.1	0.2

1 Background leakage is based on relationship in Figure 35 using average pressure in the corresponding period.
2 Toilete leakage is based on relationship yielding maximum rate in Figure 37 using average pressure in the corresponding period.

Table 5: Background leakage rates based on Eq. (2) used in current and constants suggested by Lambert et al. (1999) but with power exponents obtained in this study versus best-fit power relationships obtained in this study for the ductile and cast iron pipe sub-DMAs in Ottawa and AC pipe sub-DMA in Regina

Location, pipe type	Pressure (psi)	Background leakage rate (L/conn/h)		
		Eq. (2), with constants suggested by Lambert et al. (1999) but with power exponents obtained in this study[1]	Best-fit power relationship of measured rates (this study)[2]	Difference
Regina's AC pipe sub-DMA	28.2	1.201	1.190	-0.011
	54	1.717	1.701	-0.016
	69.2	1.967	1.949	-0.018
Ottawa's ductile iron pipe sub-DMA	53	1.420	1.665	0.245
	71.5	1.980	2.322	0.341
	92	2.620	3.071	0.451
Ottawa's, cast iron pipe sub-DMA	53	1.377	1.595	0.218
	65	1.814	2.101	0.287
	84	2.565	2.971	0.406

1 Constants A, B and C used in Eq. (2) were equal to 20 L/km/h, 1.25 L/connection/h and 0.5 L/connection/hour, respectively, and power exponents were 0.5,1.11 and 1.35 for asbestos-cement, ductile and cast iron pipes respectively.

2 Best fit relation ships were $0.2448P^{0.5}$,$0.0203P^{1.11}$ and $0.0075P^{1.35}$ for asbestos-cement, ductile and cast iron pipes, respectively, where P is pressure in psi.

Table 6: Background leakage rates based on Eq. (2) but with coefficients A, B and C modified to 24 L/km/h, 1.5 L/connection/h and 0.4 L/connection/hour, respectively, and power exponents obtained in this study versus best-fit power relationships obtained in this study for the ductile and cast iron pipe sub-DMAs in Ottawa and AC pipe sub-DMA in Regina

Location, pipe type	Background leakage rate (L/conn/h)			
	Pressure (psi)	Eq. (2), but with modified coefficients and power exponents obtained in this study[1]	Best-fit power relationship of measured rates (this study)[2]	Difference
Regina's AC pipe sub-DMA	28.2	1.321	1.190	-0.131
	54	1.888	1.701	-0.187
	69.2	2.164	1.949	-0.214
Ottawa's ductile iron pipe sub-DMA	53	1.560	1.665	0.105
	71.5	2.175	2.322	0.147
	92	2.877	3.071	0.194
Ottawa's, cast iron pipe sub-DMA	53	1.518	1.595	0.077
	65	2.000	2.101	0.102
	84	2.827	2.971	0.144

1 Constants A, B and C used in Eq. (2) were equal to 24 L/km/h, 1.5 L/connection/h and 0.4 L/connection/hour, respectively, and power exponents were 0.5,1.11 and 1.35 for asbestos-cement, ductile and cast iron pipes respectively.
2 Best fit relation ships were $0.2448P^{0.5}$,$0.0203P^{1.11}$ and $0.0075P^{1.35}$ for asbestos-cement, ductile and cast iron pipes, respectively, where P is pressure in psi.

Table 7: Effect of the exponent of the power relationship between Unavoidable Annual Real Losses (UARL), in L/connection/h, and pressure, in psi

	A	B	C	E	F	G	H	I	J
Pressure	Background leakage		Unreported & reported leaks		UARL			% ILI error	
	N1 = 1	N1 = 1.5	N1=0.5	N1 =1	(A+E)	(B+C)	(B+E)	(G/F–1)×100	(H/F–1)×100
5	0.14	0.04	0.24	0.06	0.21	0.28	0.10	34.55	-50.53
10	0.29	0.11	0.34	0.13	0.41	0.45	0.24	8.18	-42.90
15	0.43	0.20	0.42	0.19	0.62	0.62	0.39	-1.03	-37.04
20	0.57	0.31	0.48	0.26	0.83	0.79	0.56	-5.08	-32.10
25	0.71	0.43	0.54	0.32	1.04	0.96	0.75	-6.85	-27.75
30	0.86	0.56	0.59	0.39	1.24	1.15	0.95	-7.45	-23.82
35	1.00	0.71	0.64	0.45	1.45	1.34	1.16	-7.34	-20.20
40	1.14	0.86	0.68	0.51	1.66	1.54	1.38	-6.81	-16.83
45	1.29	1.03	0.72	0.58	1.86	1.75	1.61	-6.00	-13.67
50	1.43	1.21	0.76	0.64	2.07	1.97	1.85	-4.99	-10.68
55	1.57	1.39	0.80	0.71	2.28	2.19	2.10	-3.86	-7.83
60	1.71	1.59	0.83	0.77	2.49	2.42	2.36	-2.63	-5.12
65	1.86	1.79	0.87	0.84	2.69	2.66	2.63	-1.34	-2.51
70	2	2	0.90	0.90	2.90	2.90	2.90	0.00	0.00
75	2.14	2.22	0.93	0.96	3.11	3.15	3.18	1.37	2.42
80	2.29	2.44	0.96	1.03	3.31	3.41	3.47	2.76	4.76
85	2.43	2.68	0.99	1.09	3.52	3.67	3.77	4.16	7.03
90	2.57	2.92	1.02	1.16	3.73	3.94	4.07	5.57	9.23
95	2.71	3.16	1.05	1.22	3.94	4.21	4.38	6.98	11.38
100	2.86	3.41	1.08	1.29	4.14	4.49	4.70	8.39	13.46
105	3.00	3.67	1.10	1.35	4.35	4.78	5.02	9.80	15.50
110	3.14	3.94	1.13	1.41	4.56	5.07	5.35	11.21	17.49
115	3.29	4.21	1.15	1.48	4.76	5.36	5.69	12.61	19.43
120	3.43	4.49	1.18	1.54	4.97	5.67	6.03	14.00	21.33
125	3.57	4.77	1.20	1.61	5.18	5.98	6.38	15.38	23.19
130	3.71	5.06	1.23	1.67	5.39	6.29	6.73	16.76	25.02
135	3.86	5.36	1.25	1.74	5.59	6.61	7.09	18.12	26.81
140	4.00	5.66	1.27	1.80	5.80	6.93	7.46	19.48	28.57
145	4.14	5.96	1.30	1.86	6.01	7.26	7.83	20.82	30.29
150	4.29	6.27	1.32	1.93	6.21	7.59	8.20	22.16	31.99

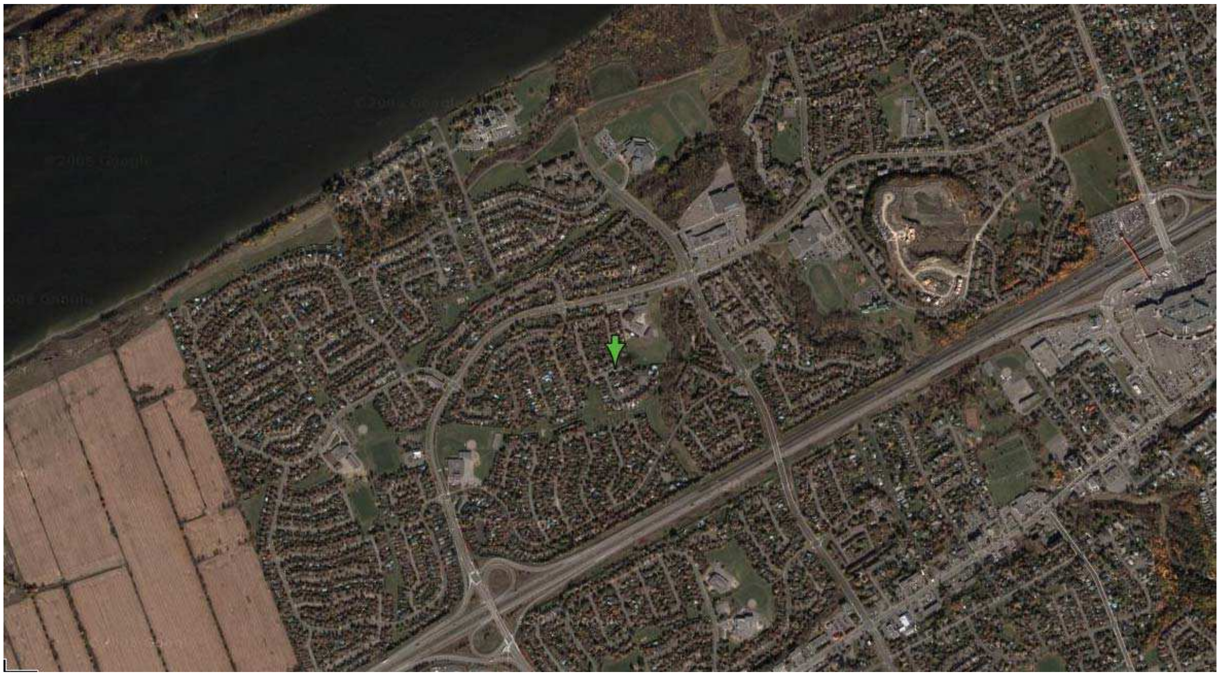

Figure 1: Aerial view of ductile iron pipe DMA in Orleans

Landuse by Parcel with duplicates removed				
NOTE: Need to recode the MLT landuse to SF along river				
		DMA Modified		
	LAND_USE	Parcels	Hectares	AssessPoints
	APT	1	2.59	240
	MLT	346	17.04	
	SF	1431	77.05	
	Vac_Res	24	11.32	
	Rea	24	20.29	
	Comm	3	9.84	
	Inst	5	23.01	
	Industrial	0	0.00	
	Total	1834	161.14	2073

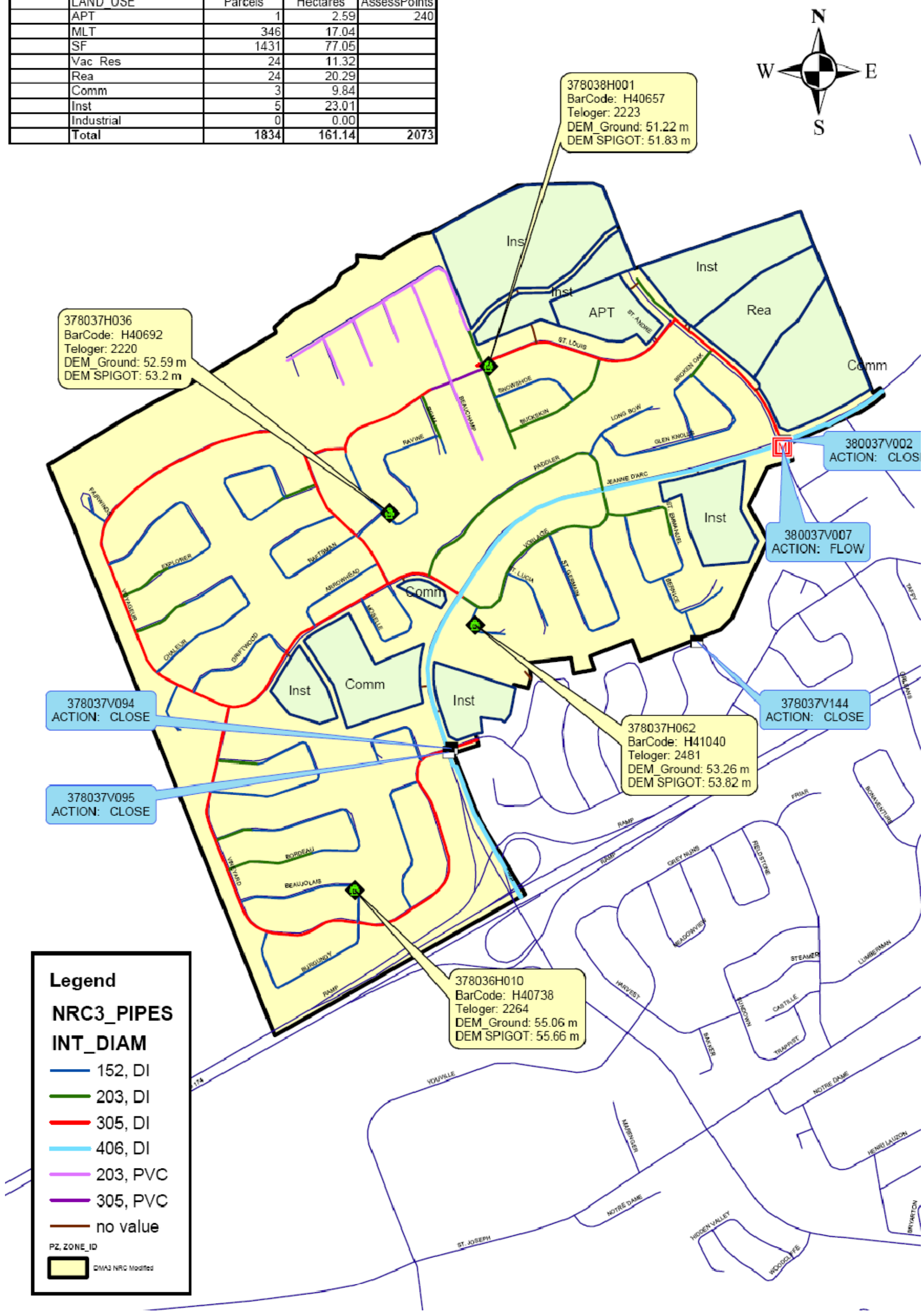

Figure 2: Boundaries of ductile iron pipe DMA in Orleans

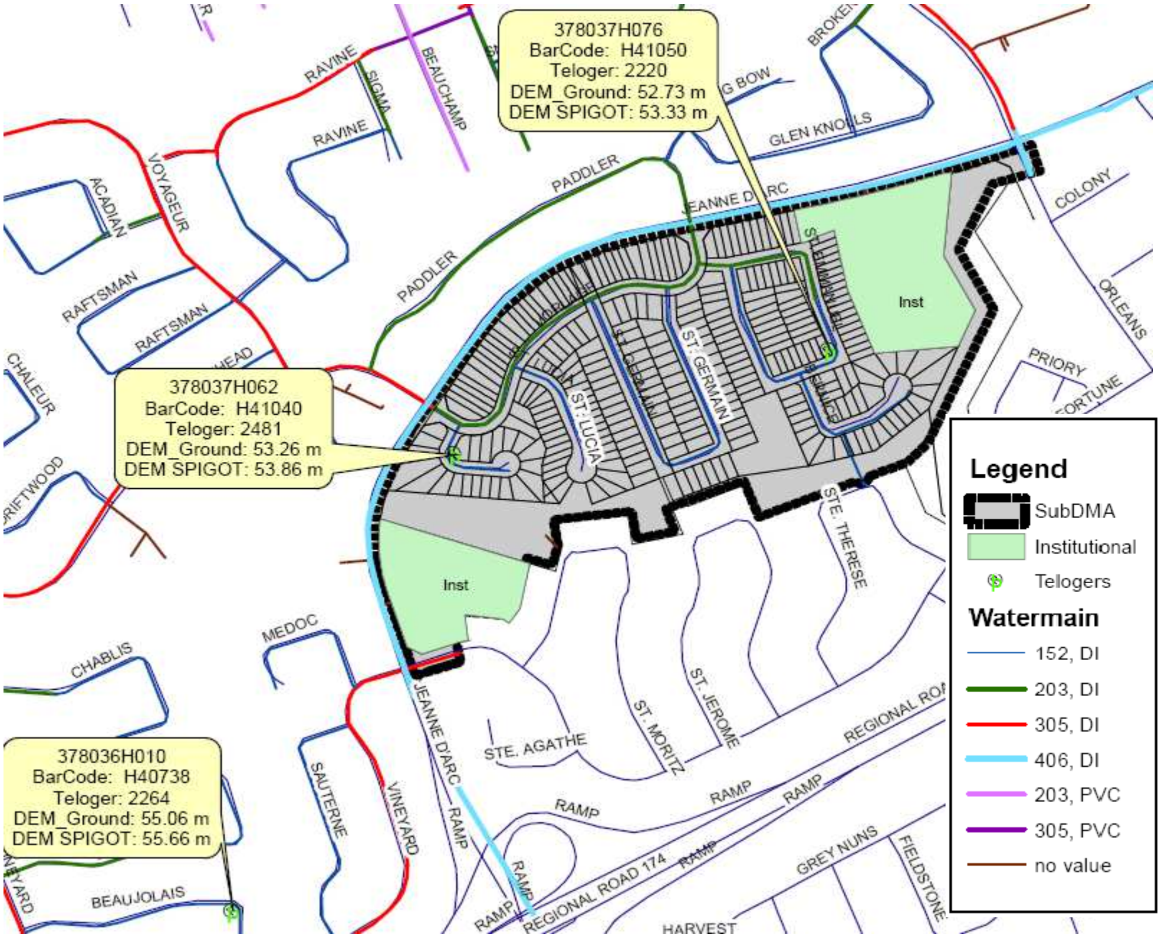

Figure 3: Boundaries of ductile iron pipe sub-DMA in Orleans

Figure 4: Aerial view of cast iron pipe DMA in Meadowlands

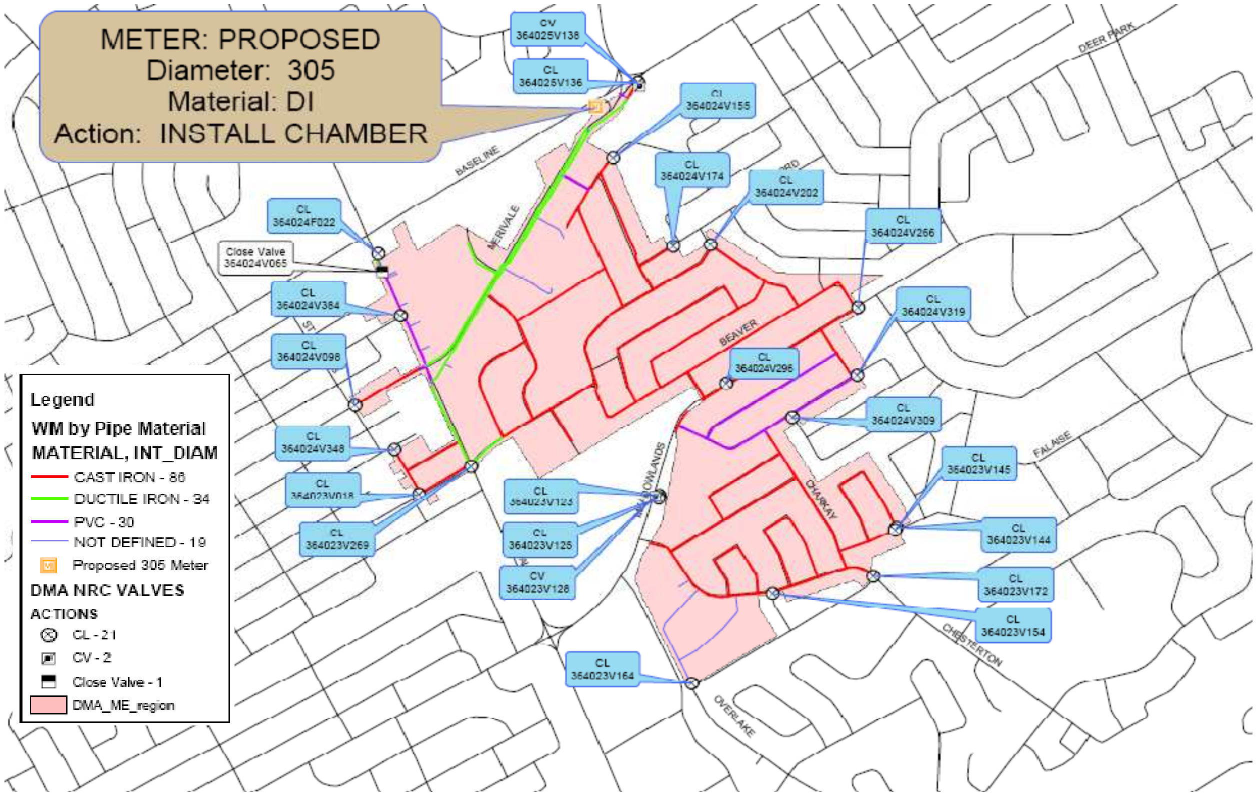

Figure 5: Boundaries of cast iron pipe DMA in Meadowlands

Missing

Figure 6: Boundaries of cast iron pipe sub-DMA in Meadowlands

Figure 7: Bypass at DMA around a closed valve at DMA inlet

Figure 8: Aboveground flow meter and PRV rig

Figure 9: Listening for acoustic noise of passing water at DMA boundary valves

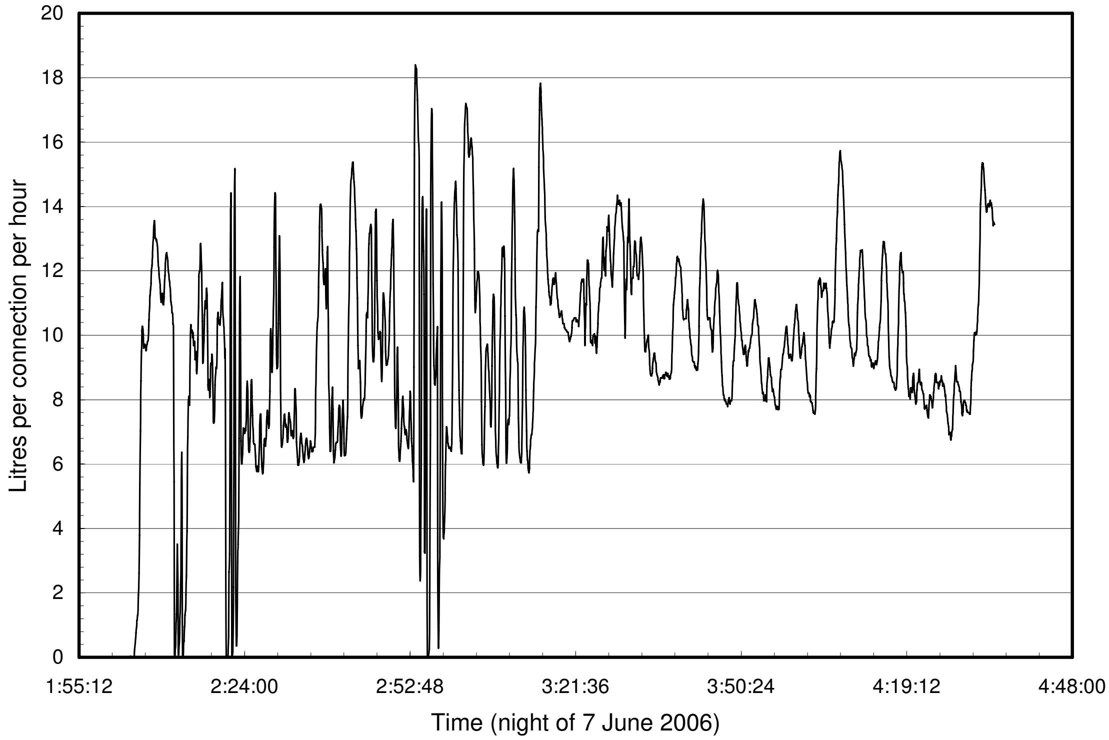

Figure 10: Flow nightline at 5-seconds interval for ductile iron sub-DMA in Orleans on night of 7 June 2006

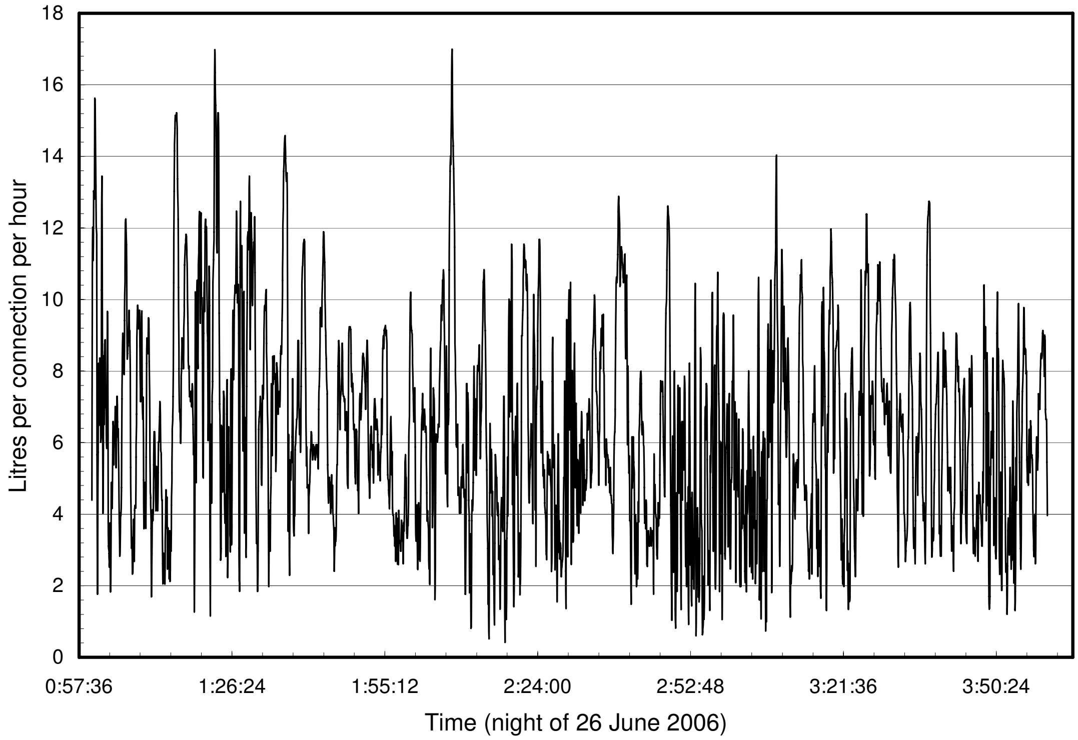

Figure 11: Flow nightline at 5-seconds interval for ductile iron sub-DMA in Orleans on night of 26 June 2006

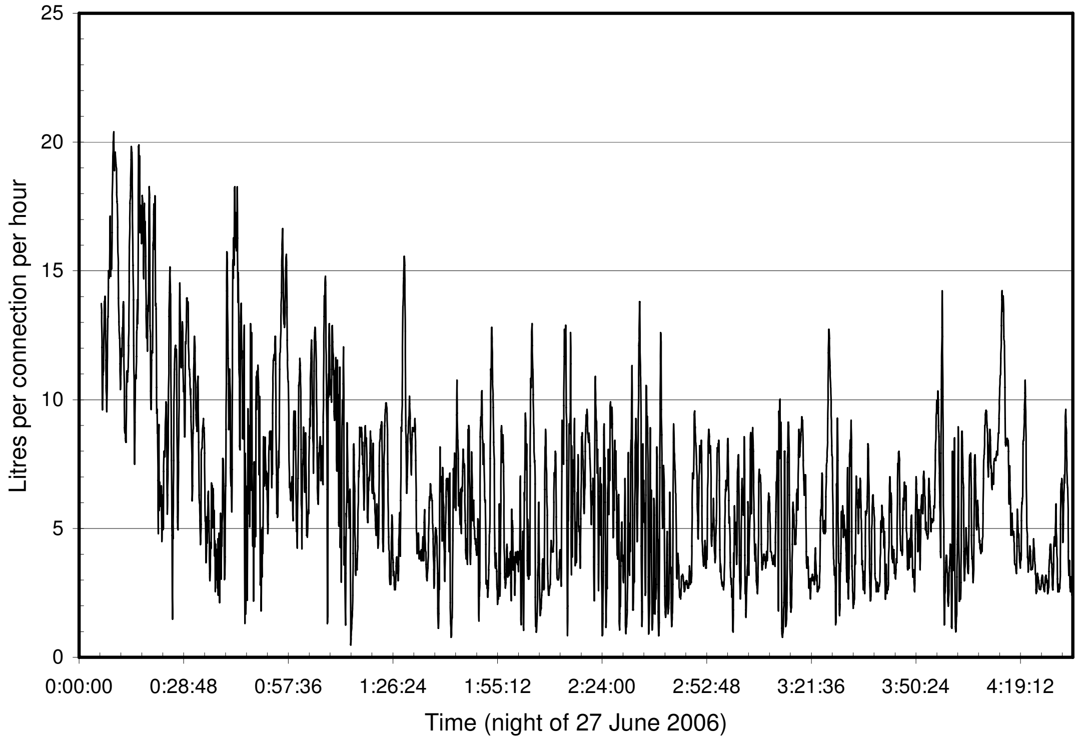

Figure 12: Flow nightline at 5-seconds interval for ductile iron sub-DMA in Orleans on night of 27 June 2006

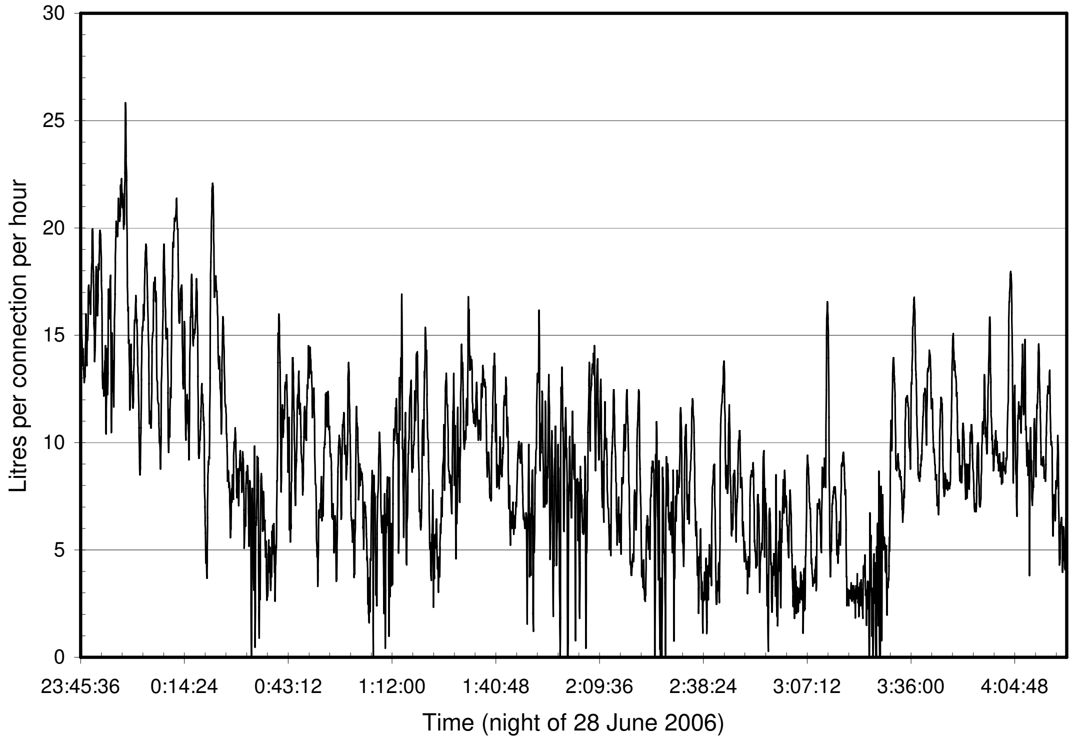

Figure 13: Flow nightline at 5-seconds interval for ductile iron sub-DMA in Orleans on night of 28 June 2006

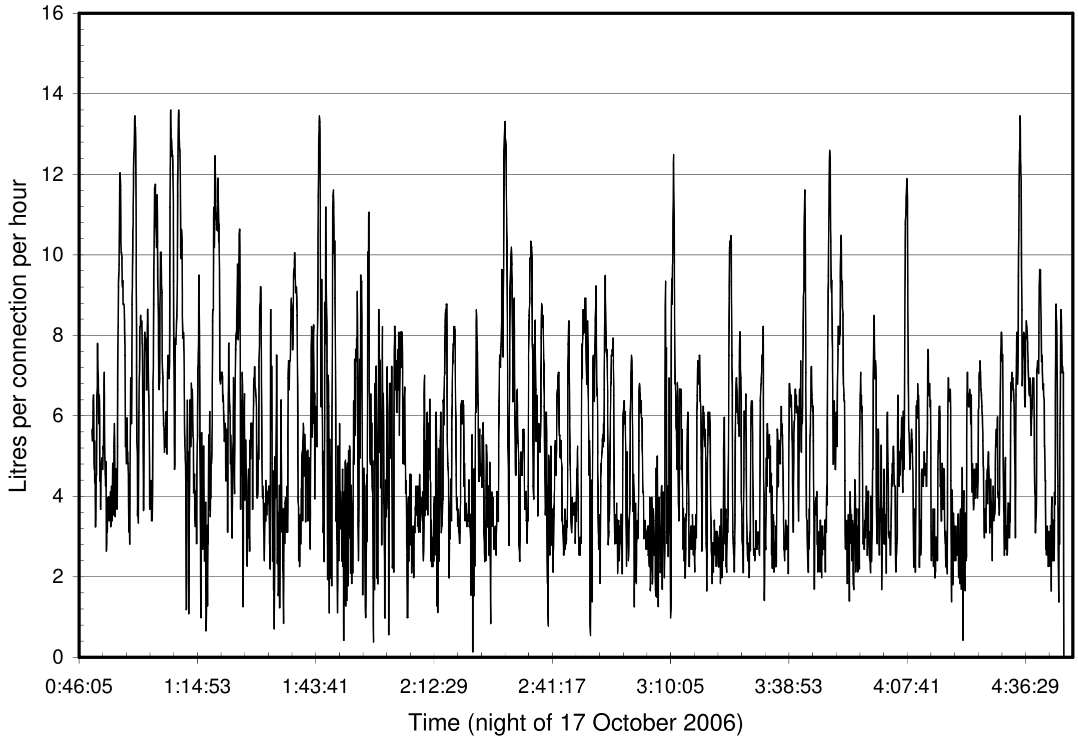

Figure 14: Flow nightline at 5-seconds interval for ductile iron sub-DMA in Orleans on night of 17 October 2006

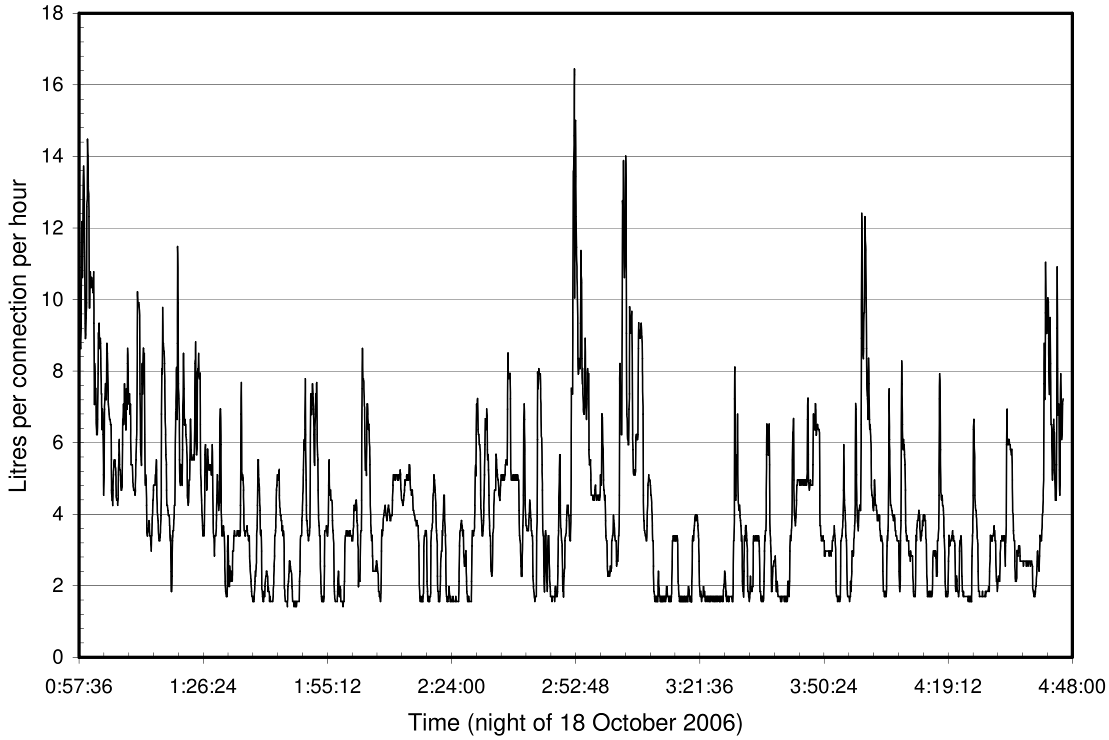

Figure 15: Flow nightline at 5-seconds interval for ductile iron sub-DMA in Orleans on 18 October 2006

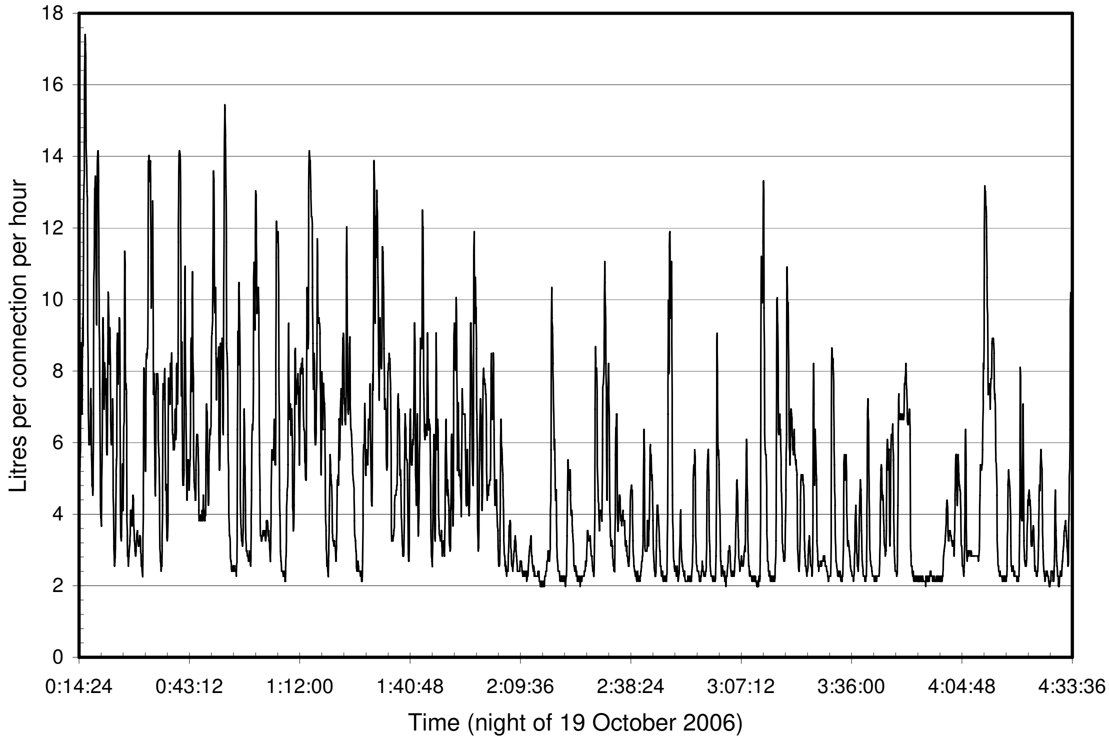

Figure 16: Flow nightline at 5-seconds interval for ductile iron sub-DMA in Orleans on night of 19 October 2006

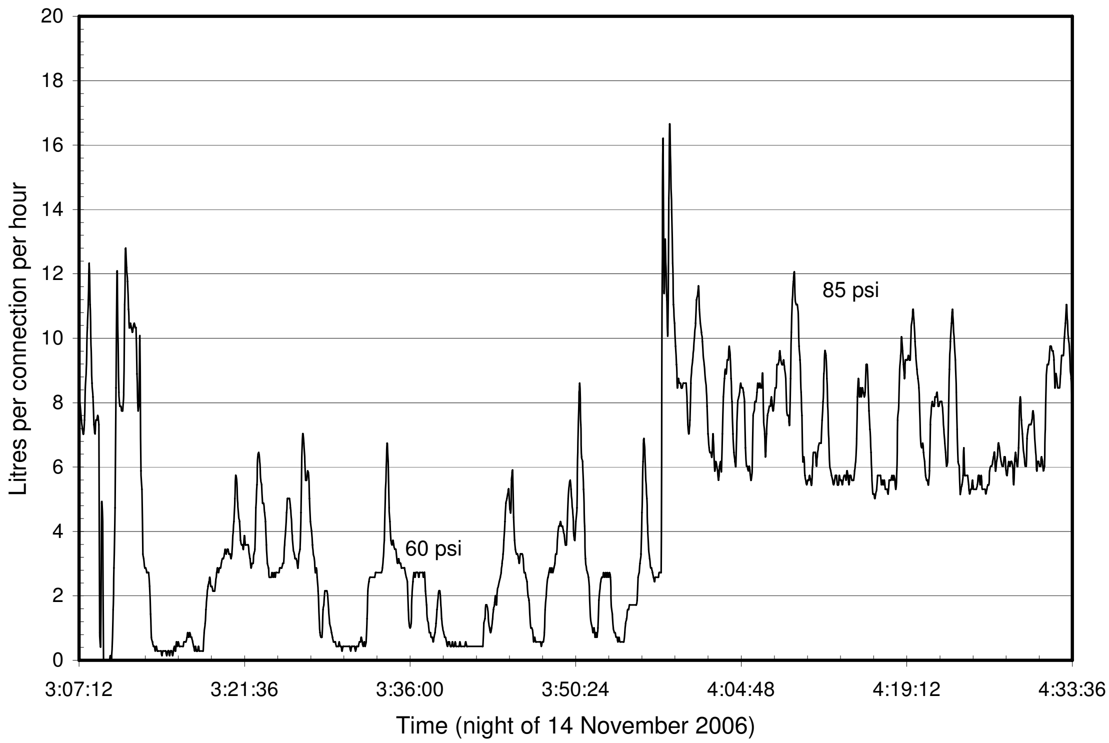

Figure 17: Flow nightline at 5-seconds interval for cast iron sub-DMA in Meadowlands on night of 14 November 2006

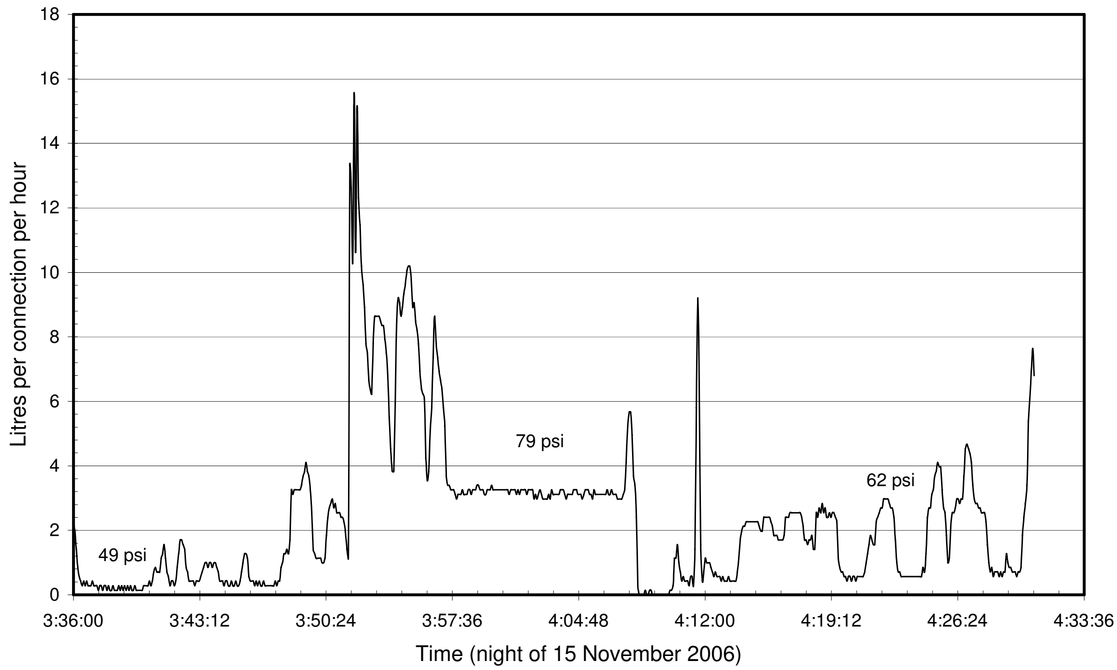

Figure 18: Flow nightline at 5-seconds interval for cast iron sub-DMA in Meadowlands on night of 15 November 2006

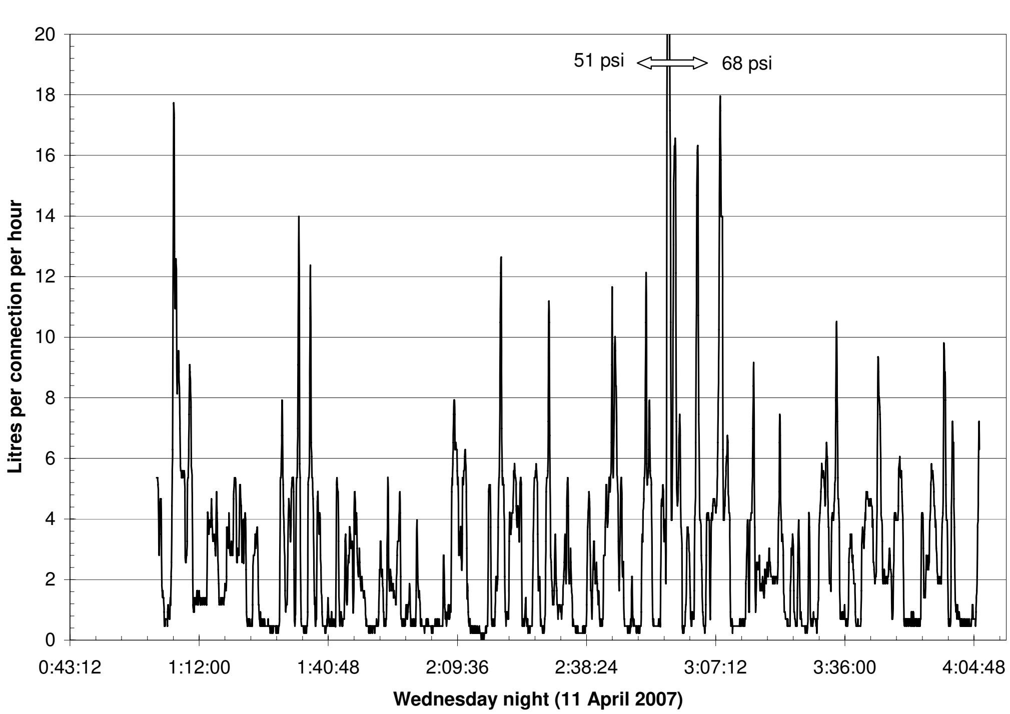

Figure 19: Flow nightline at 5-seconds interval for cast iron sub-DMA in Meadowlands on night of 11 April 2007

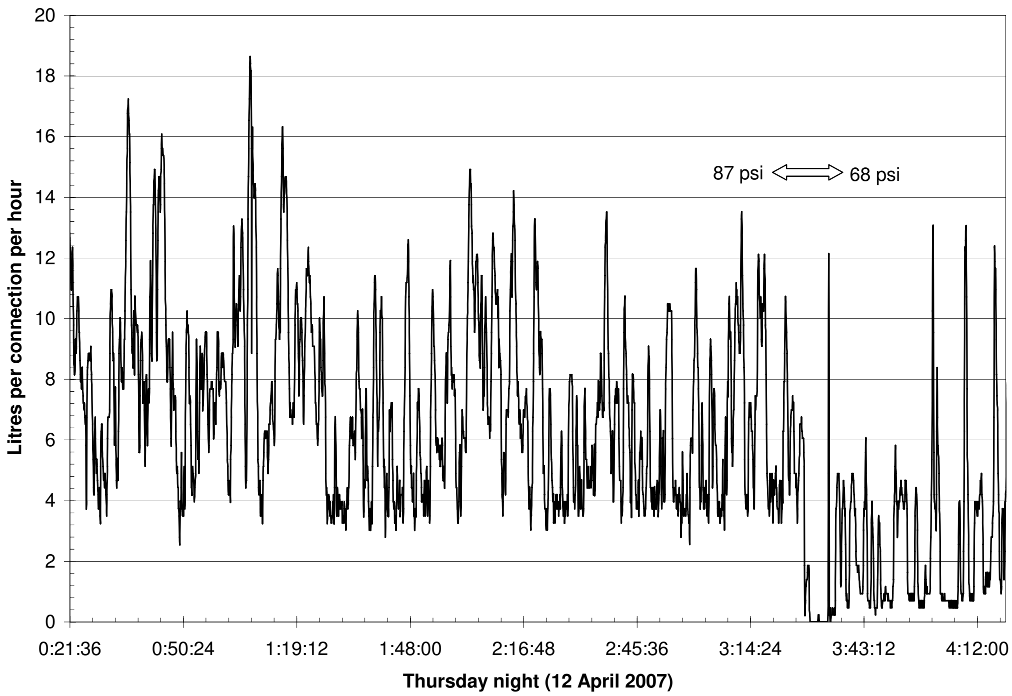

Figure 20: Flow nightline at 5-seconds interval for cast iron sub-DMA in Meadowlands on night of 12 April 2007

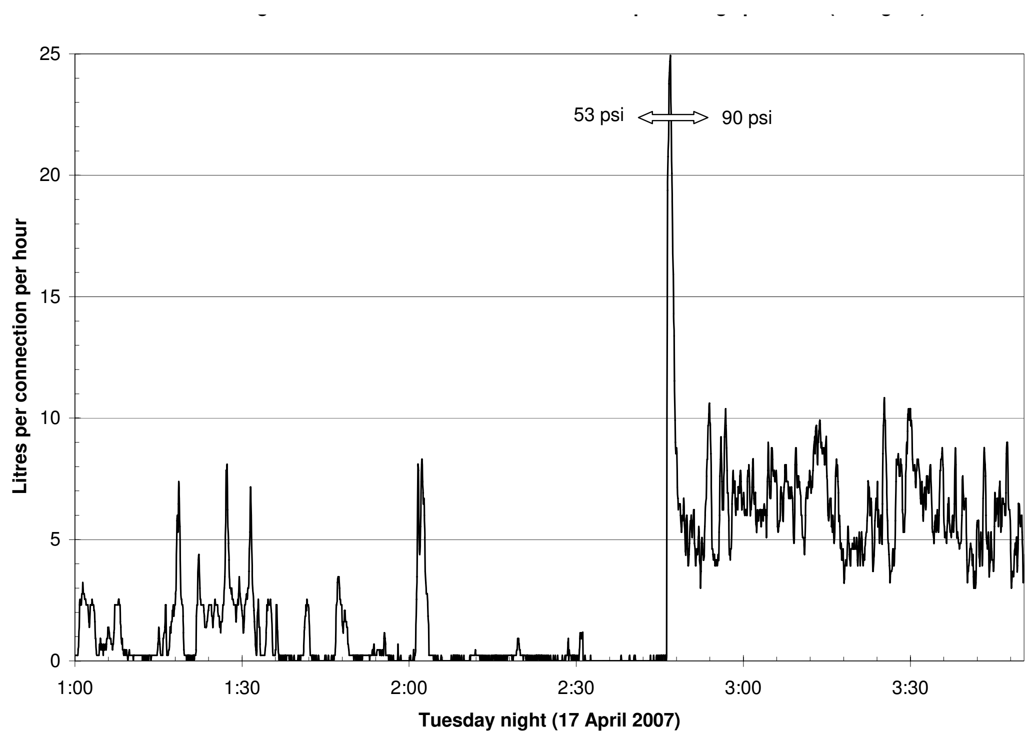

Figure 21: Flow nightline at 5-seconds interval for cast iron sub-DMA in Meadowlands on night of 17 April 2007

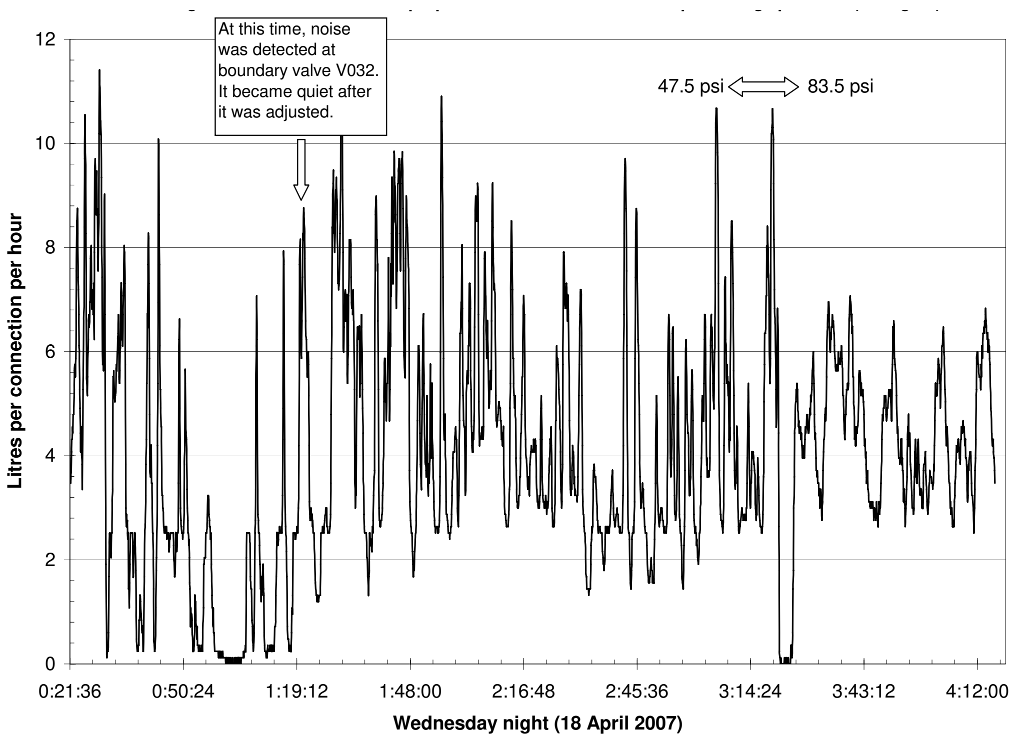

Figure 22: Flow nightline at 5-seconds interval for cast iron sub-DMA in Meadowlands on night of 18 April 2007

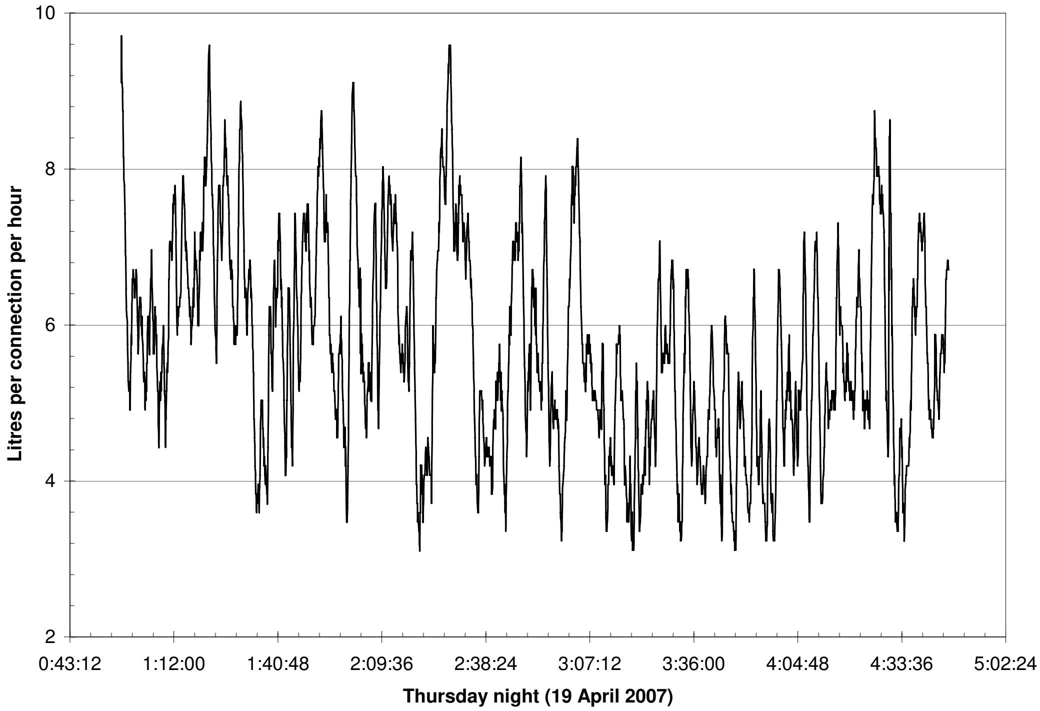

Figure 23: Flow nightline at 5-seconds interval for cast iron sub-DMA in Meadowlands on night of 19 April 2007

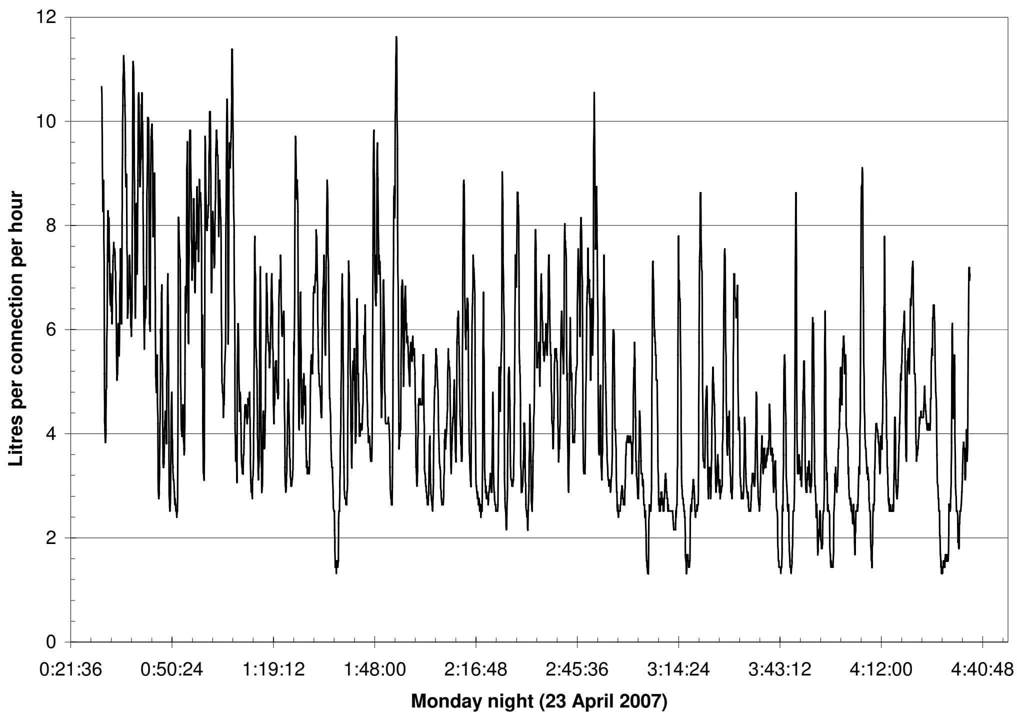

Figure 24: Flow nightline at 5-seconds interval for cast iron sub-DMA in Meadowlands on night of 23 April 2007

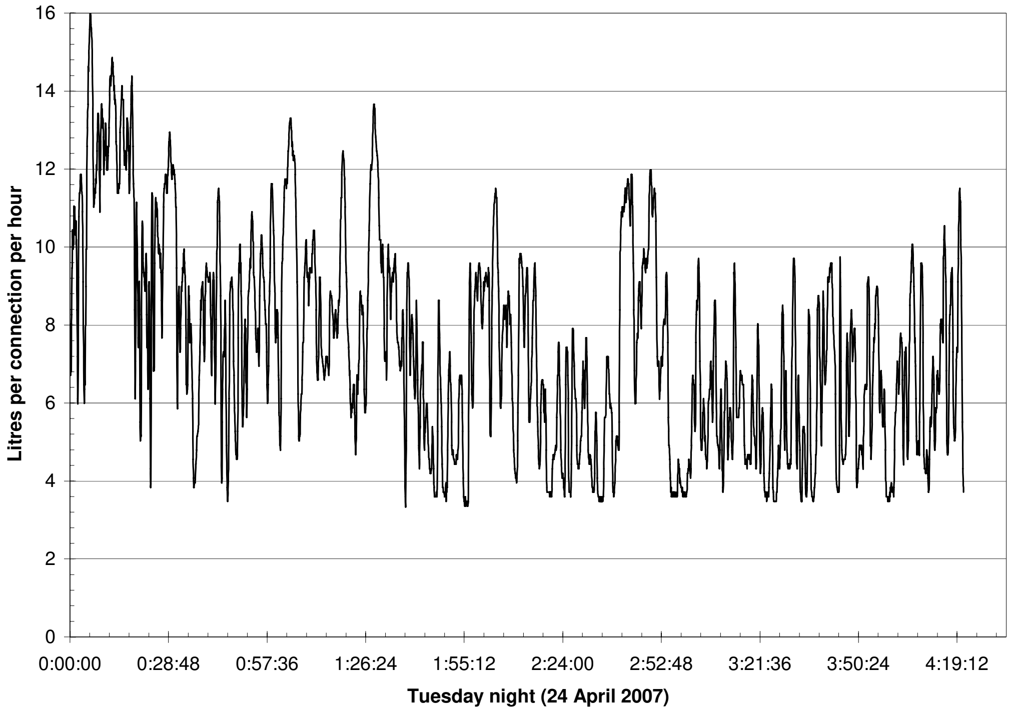

Figure 25: Flow nightline at 5-seconds interval for cast iron sub-DMA in Meadowlands on night of 24 April 2007

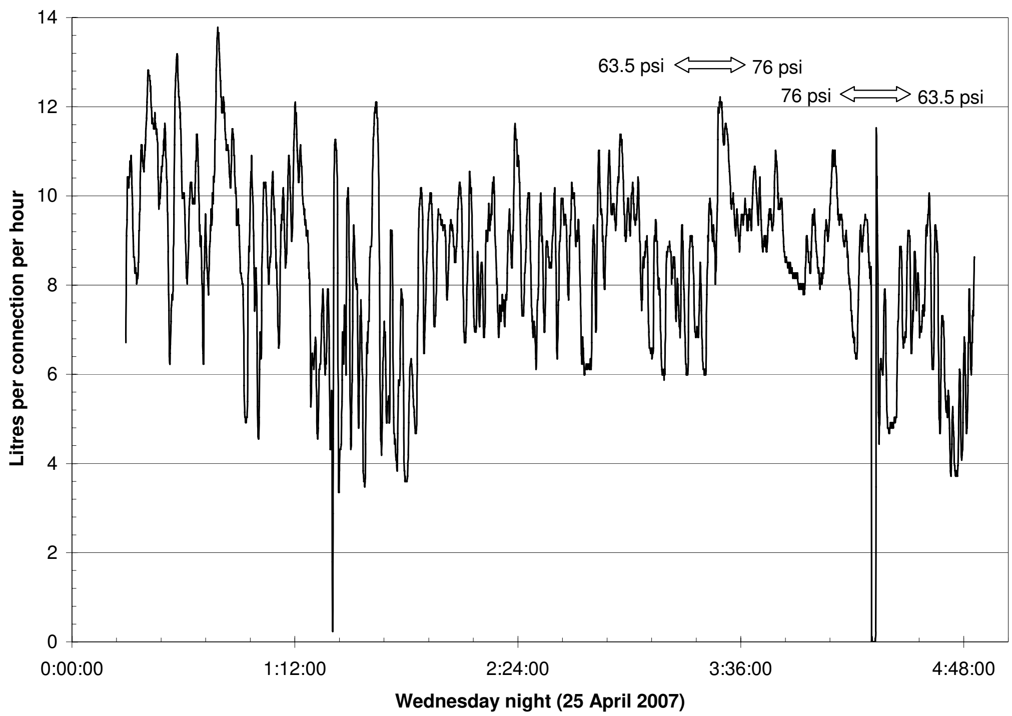

Figure 26: Flow nightline at 5-seconds interval for cast iron sub-DMA in Meadowlands on night of 25 April 2007

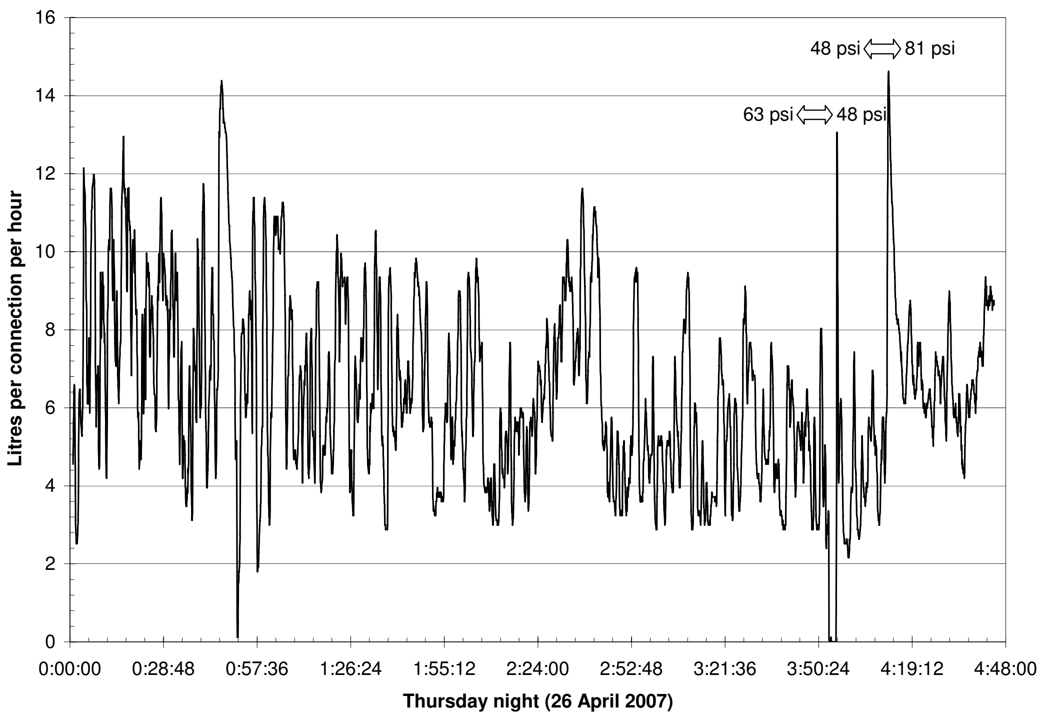

Figure 27: Flow nightline at 5-seconds interval for cast iron sub-DMA in Meadowlands on night of 26 April 2007

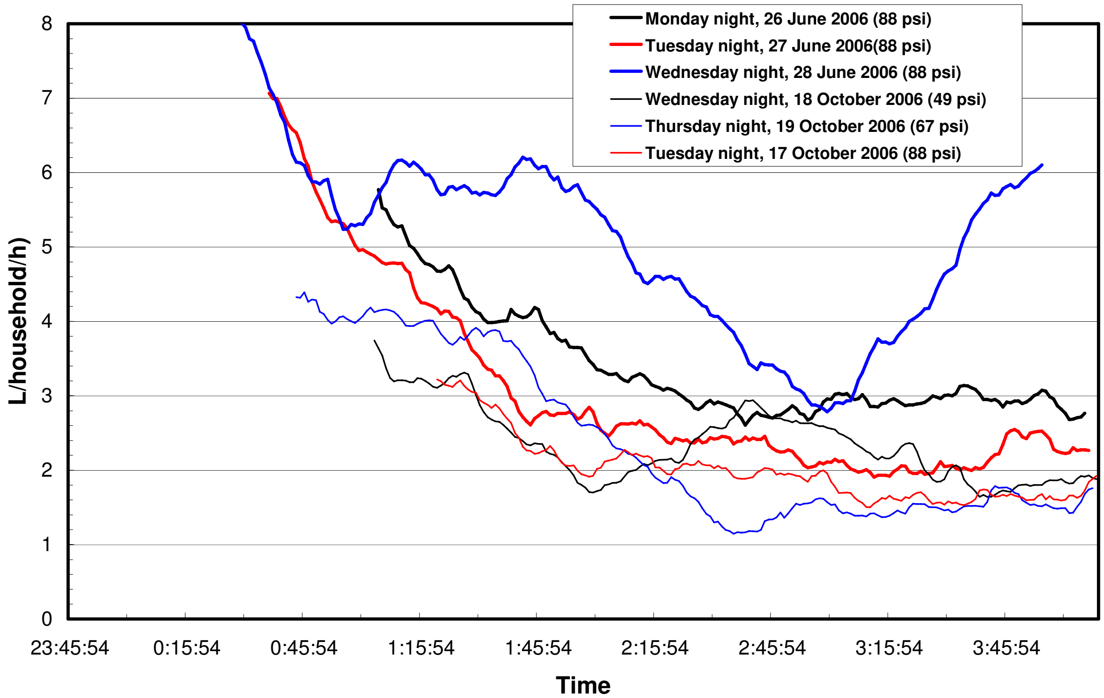

Figure 28: Moving 60-minute average residential night demand in the ductile iron pipe sub-DMA in Orleans

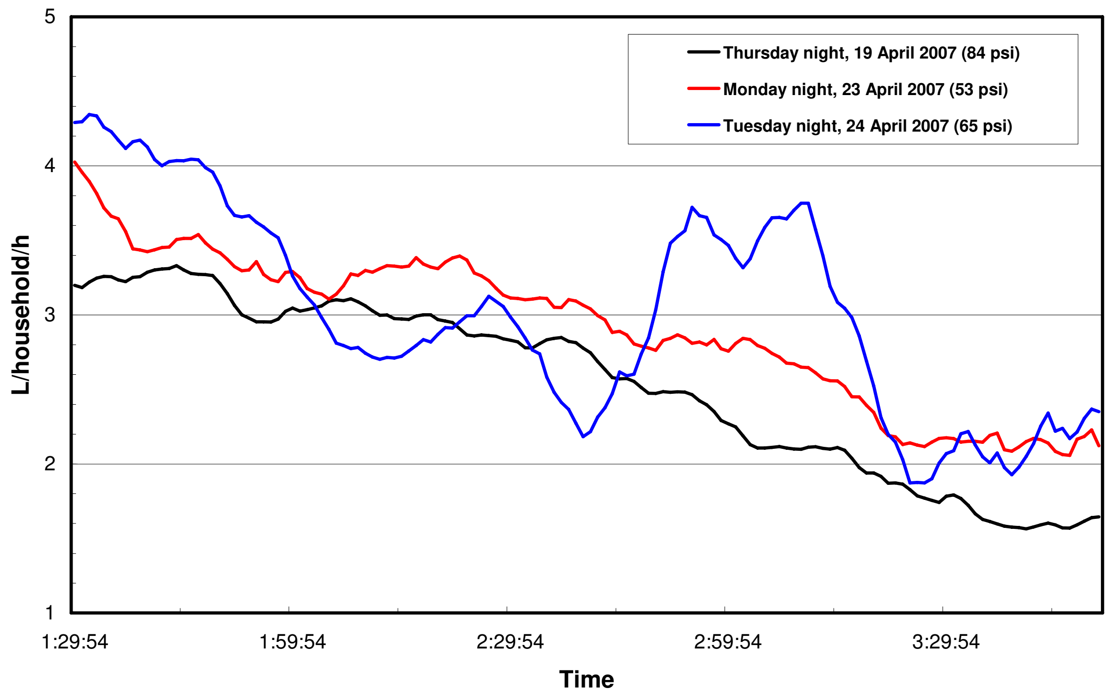

Figure 29: Moving 60-minute average residential night demand in the cast iron pipe sub-DMA in Meadowlands

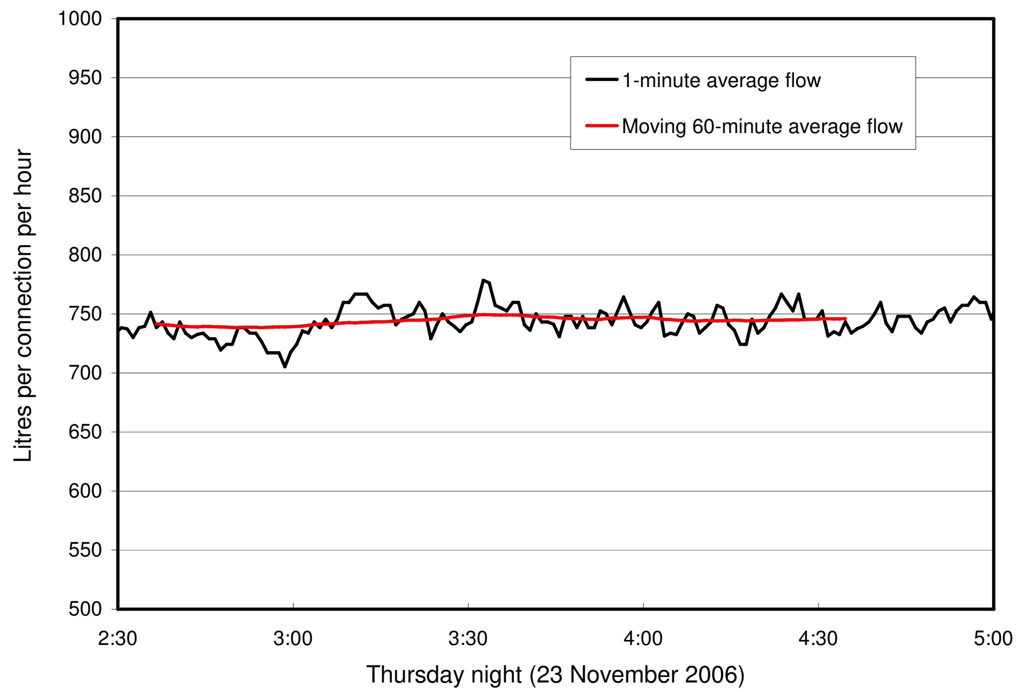

Figure 30: Flow nightline in the cast iron pipe DMA in Meadowlands on 23 November 2006

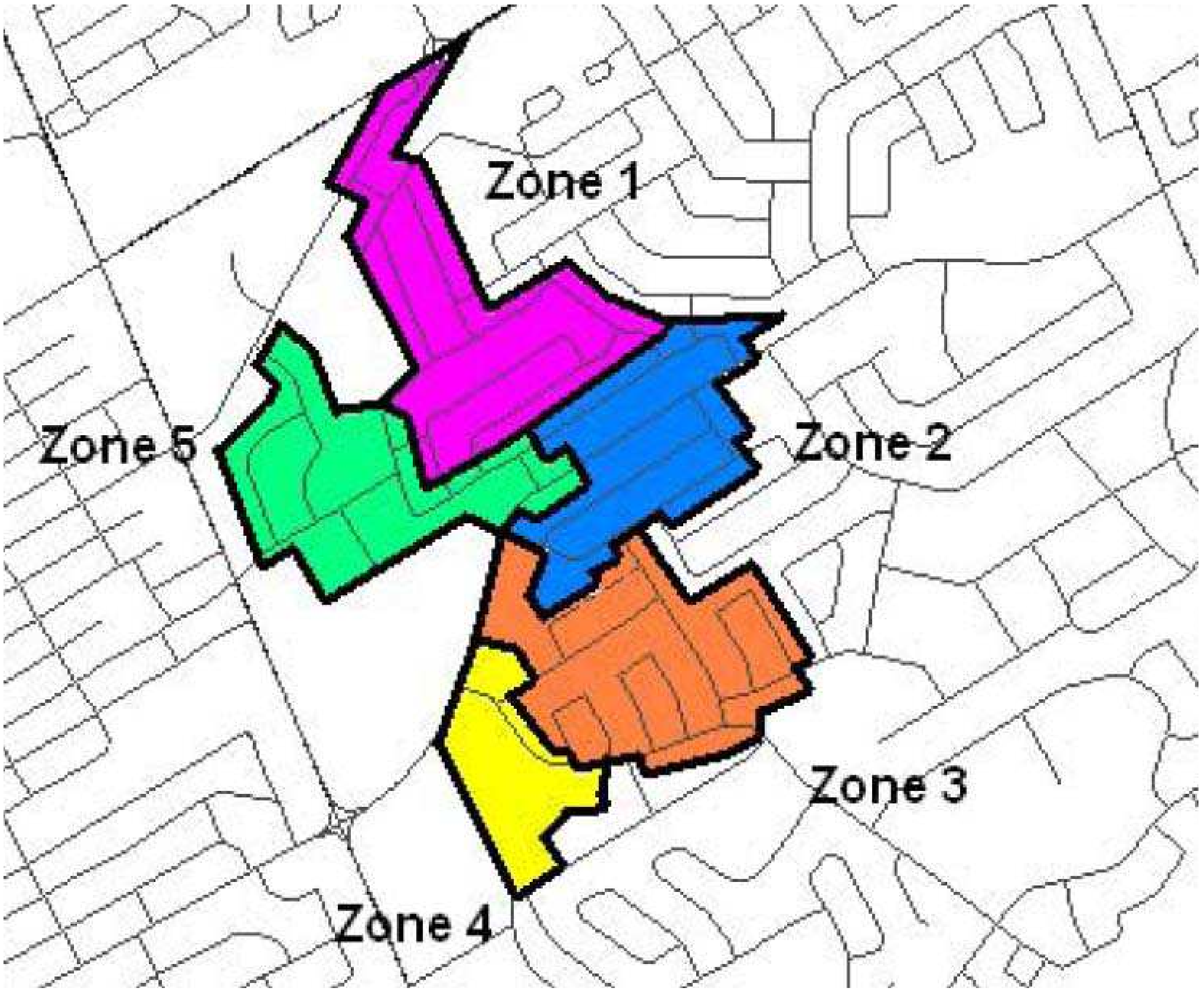

Figure 31: Zones used for flow step testing in the cast iron pipe
DMA in Meadowlands

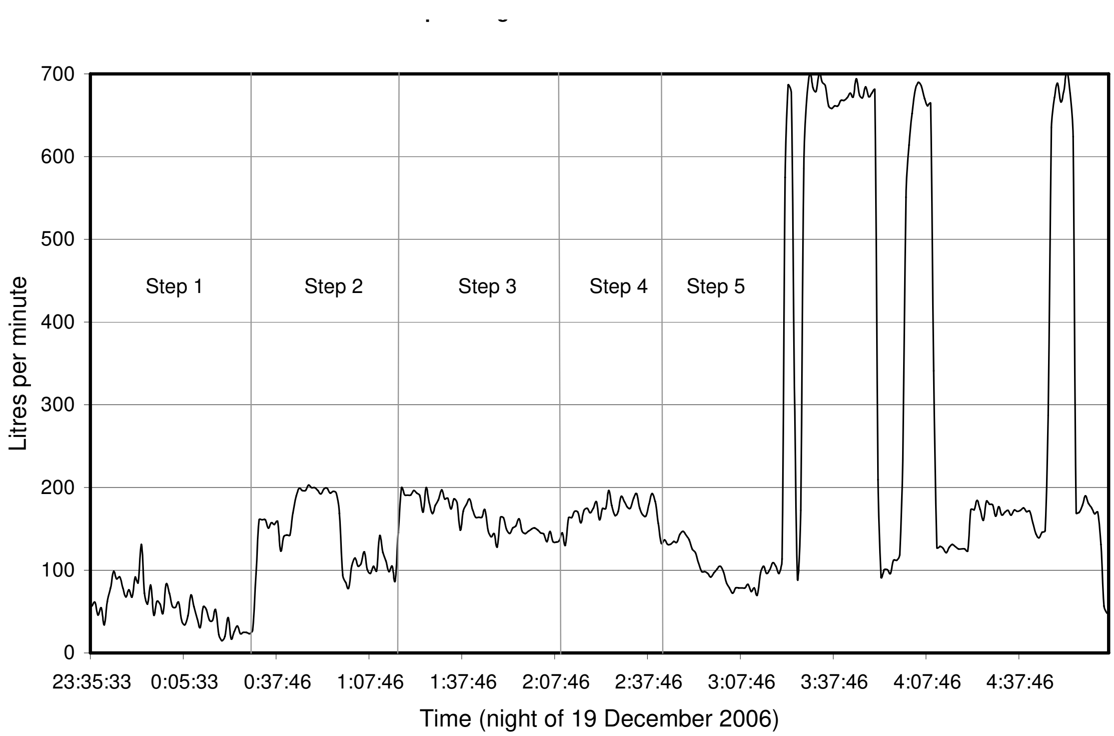

Figure 32: Flow nightline of step test in the cast iron pipe DMA in Meadowlands on 19 December 2006

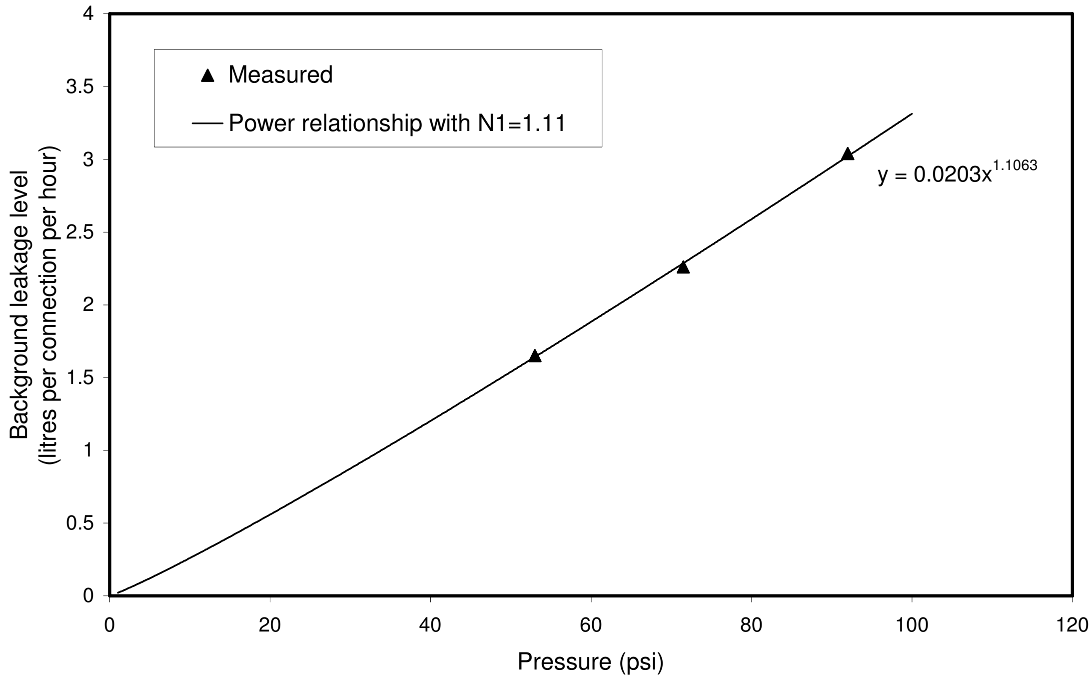

Figure 33: Background leakage rate versus pressure for ductile iron pipe sub-DMA in Orleans

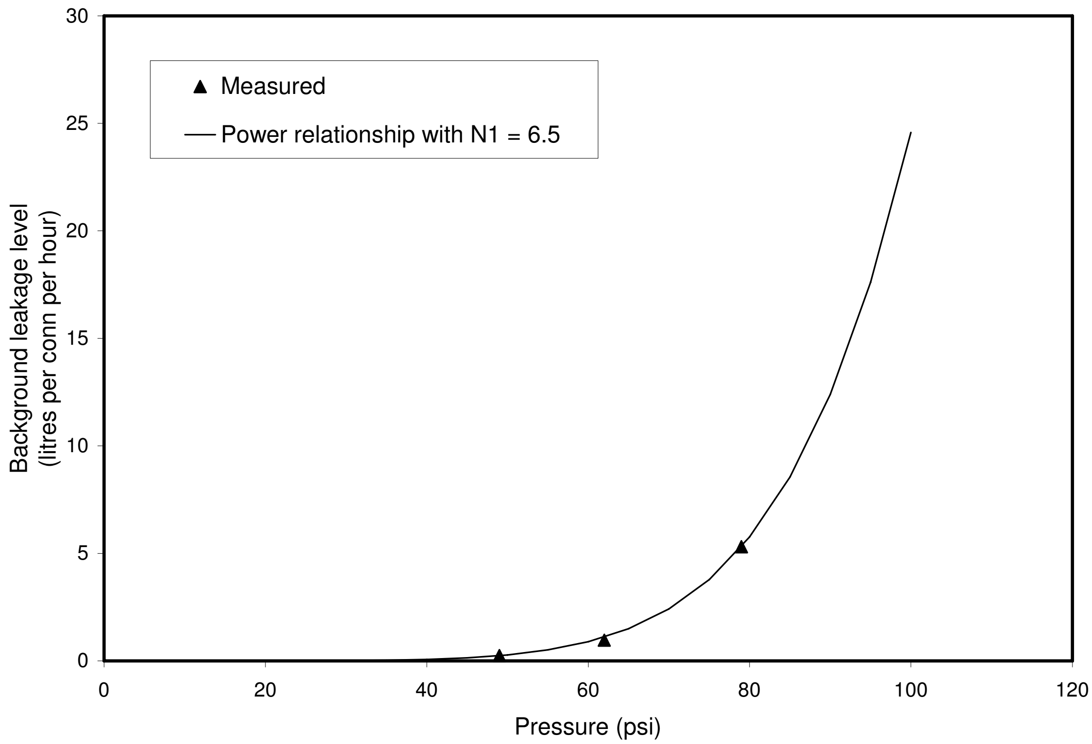

Figure 34: Background leakage rate versus pressure for cast iron pipe sub-DMA in Meadowlands

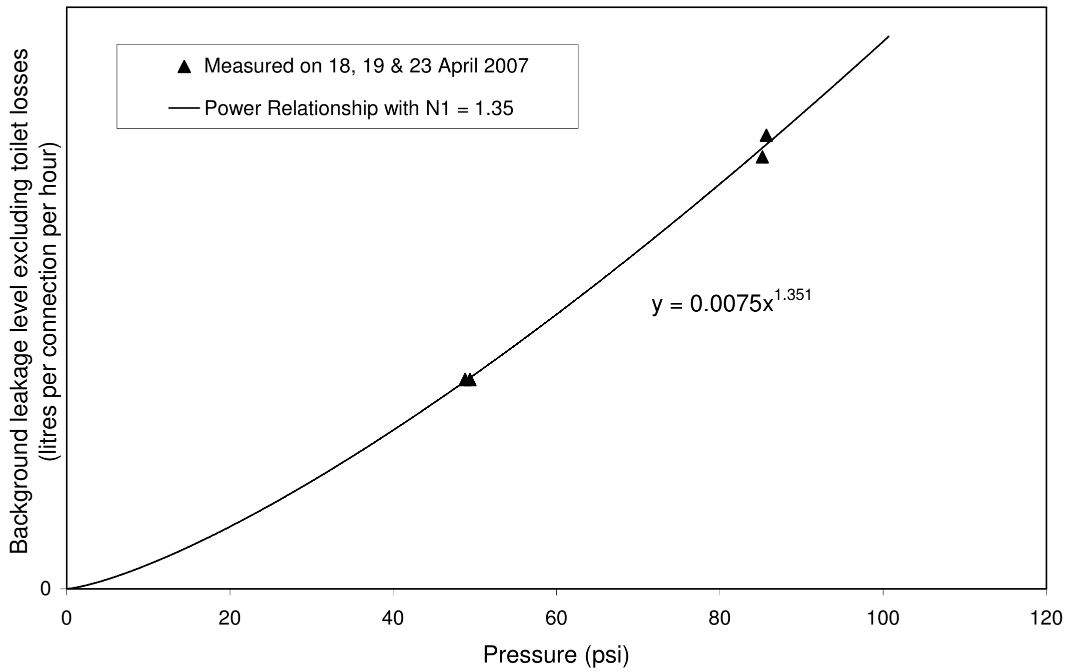

Figure 35: Background leakage rate excluding toilet losses versus pressure for cast iron pipe sub-DMA in Meadowlands

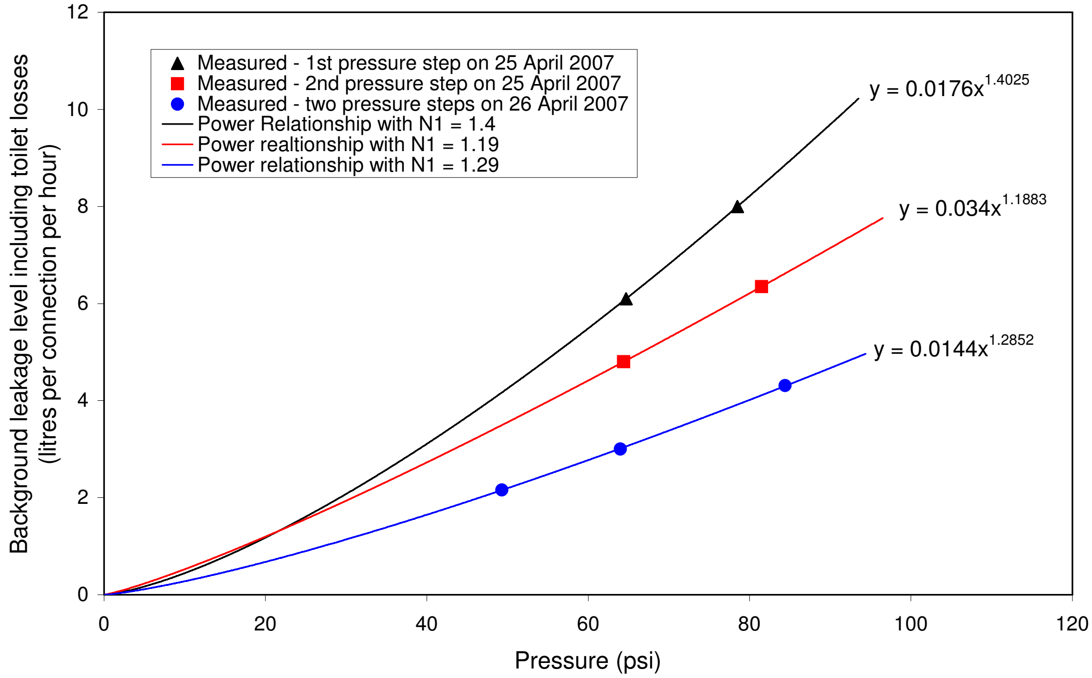

Figure 36: Background leakage rate including toilet losses versus pressure for cast iron pipe sub-DMA in Meadowlands

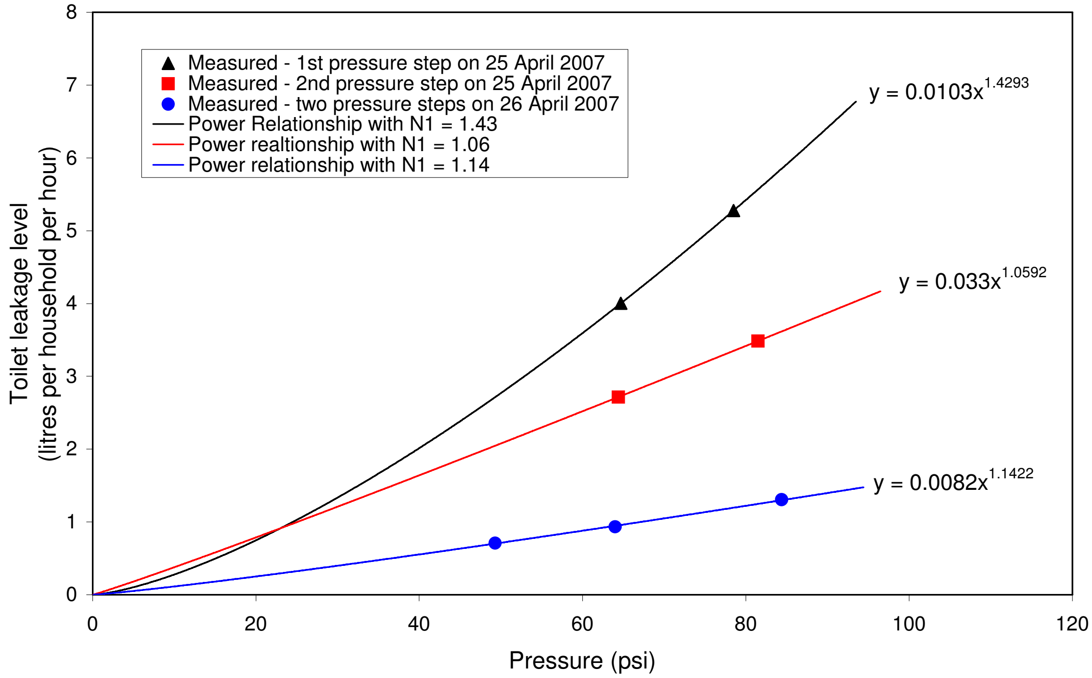

Figure 37: Toilet leakage rate versus pressure for cast iron pipe sub-DMA in Meadowlands

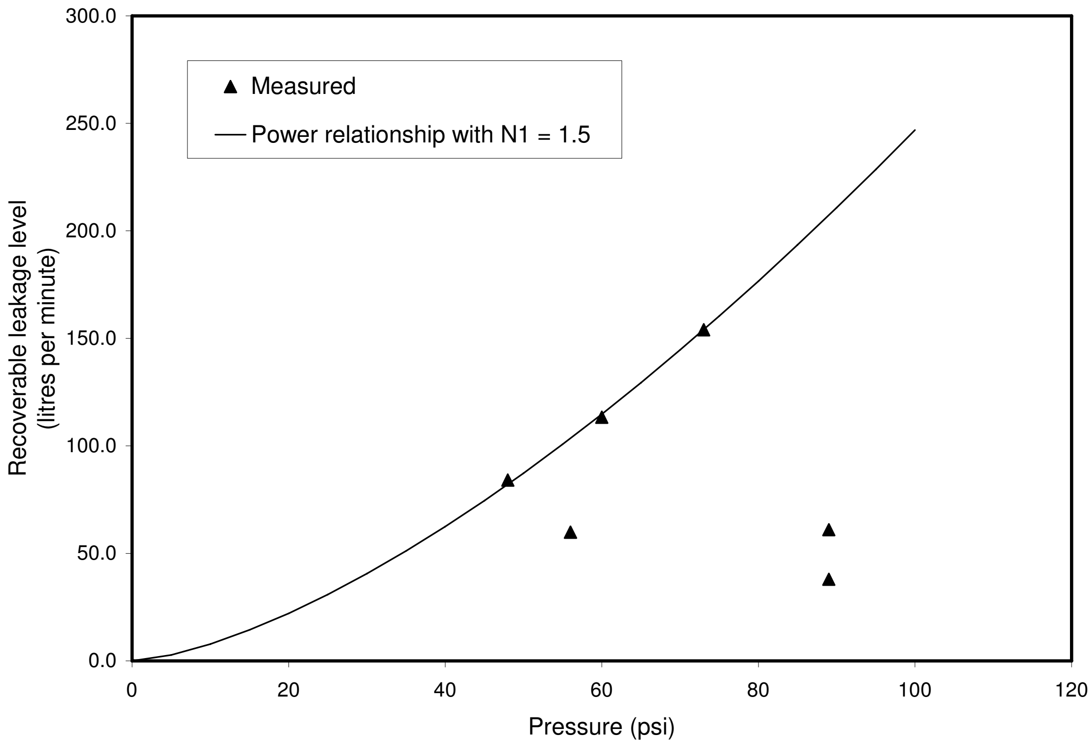

Figure 38: Recoverable leakage rate versus pressure for ductile iron pipe DMA in Orleans

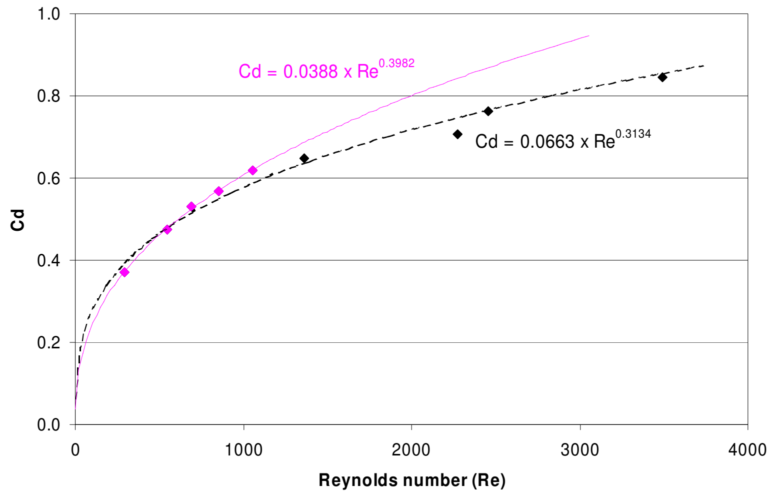

Figure 39: Discharge coefficient, C_d, versus Reynolds number for a 1-mm diameter orifice (partly reproduced from Lambert, 2001)

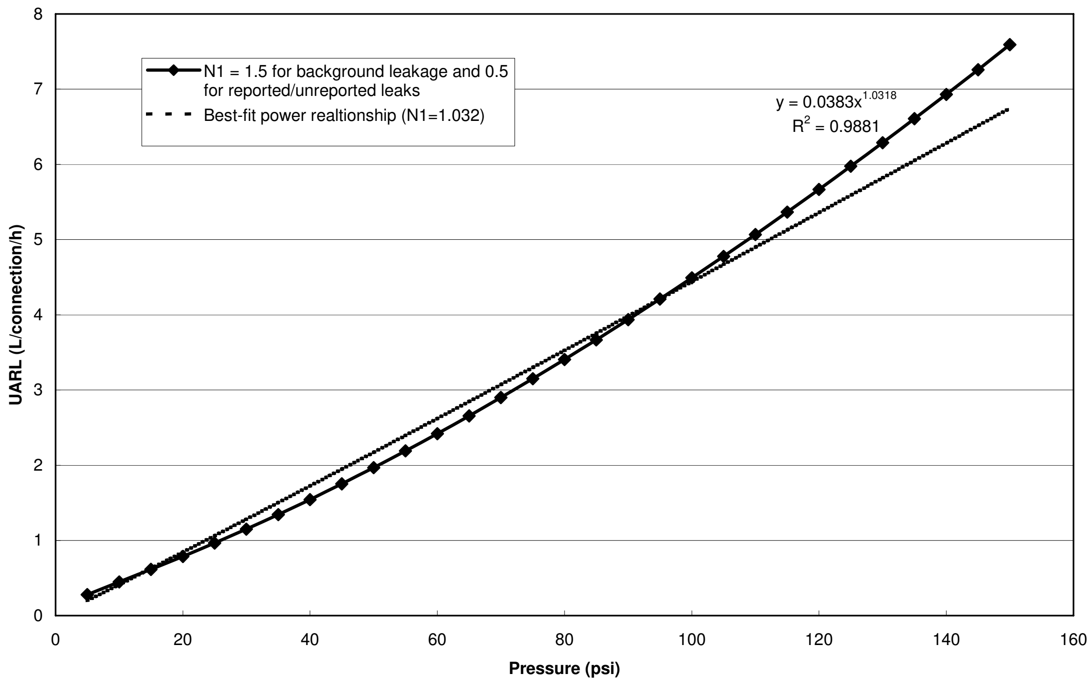

Figure 40: N1 for best-fit power relationship between pressure and UARL calculated using N1 = 1.5 and 0.5 for background and Unreported/reported leaks, respectively

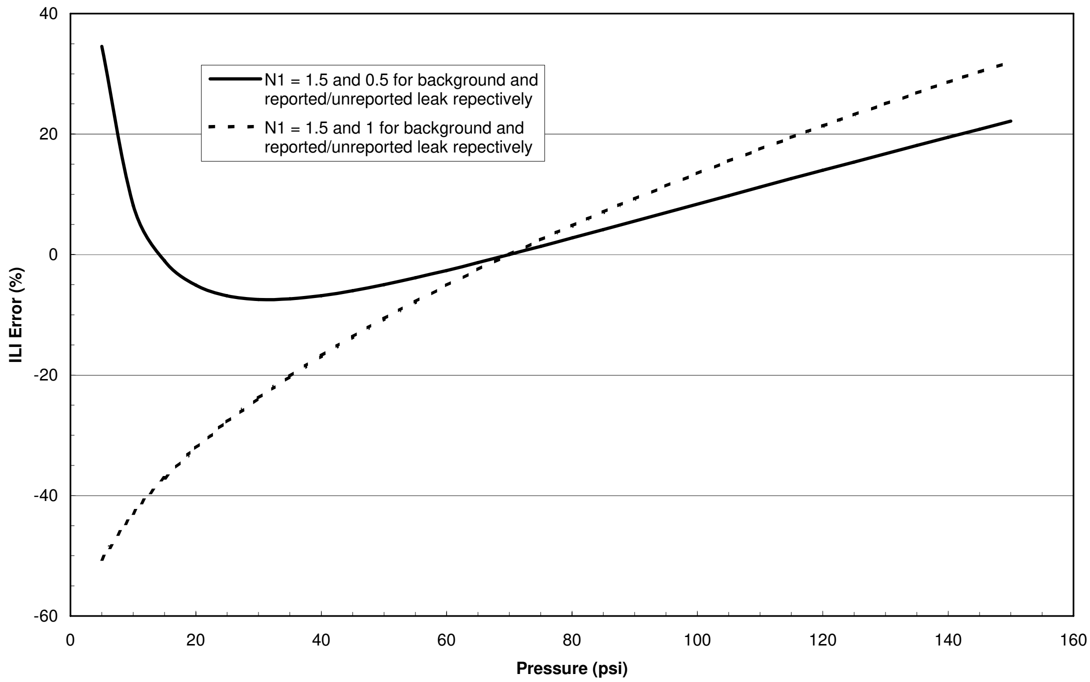

Figure 41: Error in ILI due to the assumption of a linear pressure-UARL relationship

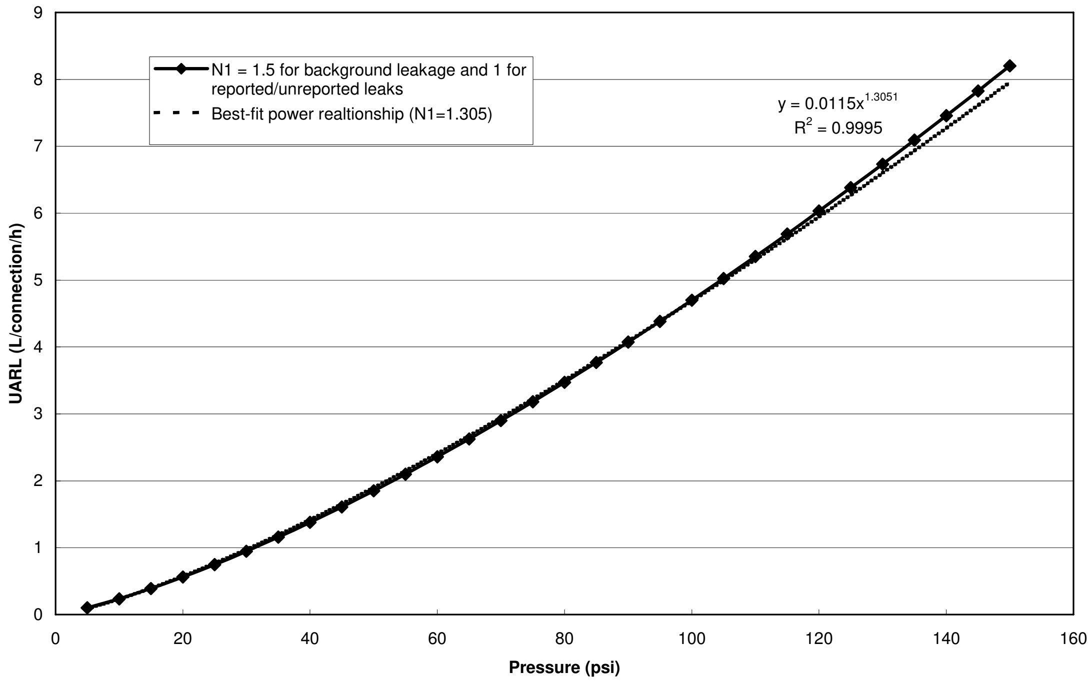

Figure 42: N1 for best-fit power relationship between pressure and UARL calculated using N1 = 1.5 and 1 for background and Unreported/reported leaks, respectively

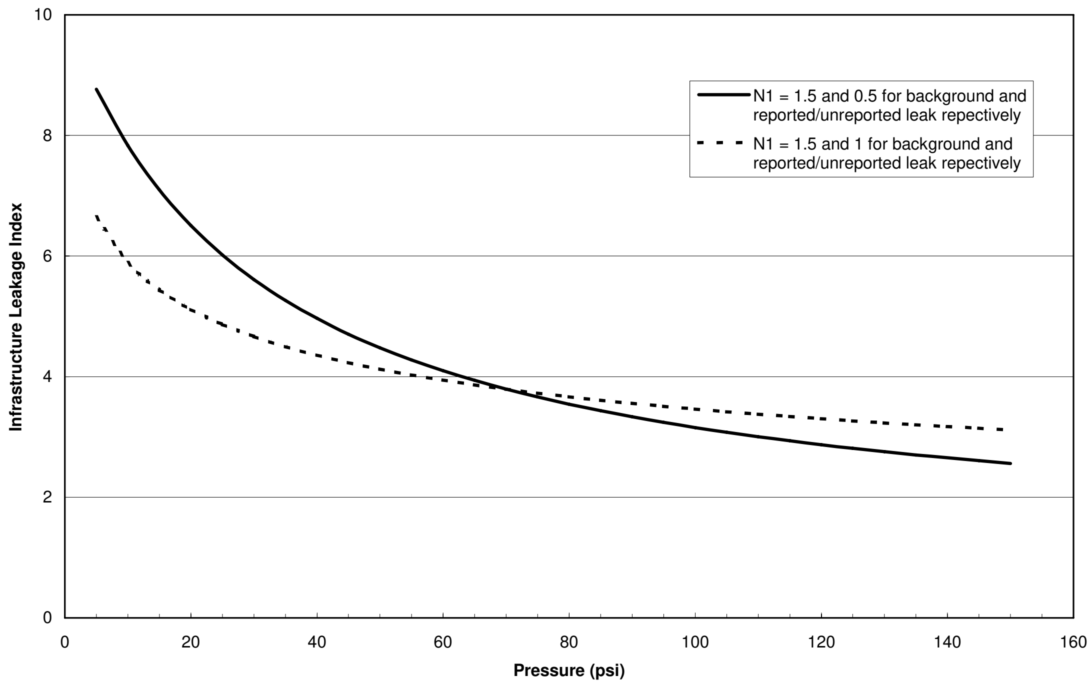

Figure 43: Variation of infrastructure leakage index with pressure for a system having a service connection density of 80 connections/km and an average service pipe length of 15 metres from curbstops to customer meters

APPENDIX A

ACOUSTIC LEAK DETECTION STRATEGIES

INTRODUCTION

Hidden or unreported leaks in water distribution pipes are normally detected by surveying pipes for the acoustic noise generated by water as it leaks from pipes under pressure. All areas of the pipe network are surveyed whether or not leakage is suspected. However, if the pipe network is divided into district-metered areas, only areas that exhibit high leakage rates or exceed a certain leakage threshold are surveyed for leaks.

Several methods or strategies can be employed for acoustic leak detection. These include manual acoustic surveys using listening equipment or noise correlators and automatic surveys using acoustic noise loggers. Adoption of a particular method depends on available resources, both financial and human; characteristics of the pipe network; and operating conditions.

A description of the above methods, discussion of the pros and cons of their operation and recent experiences with their performance is presented in this appendix. In-pipe acoustic methods that are mainly used for large-diameter transmission mains are not addressed.

LISTENING SURVEYS

Listening surveys are conducted by experienced leak inspectors, systematically working their way around the pipe network and using listening devices at appropriate pipe fittings to detect the characteristic hissing sound generated by water leaking under pressure. General listening surveys involve listening at only convenient fittings, typically fire hydrants. Because of the large spacing between listening points, general surveys may miss many small leaks on distribution and service connection pipes.

On the other hand, detailed surveys involve listening at most pipe fittings, including curbstops, and hence they can detect more leaks than general surveys. In areas of the pipe network having heavy road traffic and (or) daytime water use, leak sounds will likely be masked by interfering noise and hence may not be discerned.

Therefore, these areas are usually surveyed during the night when interference sources subside.

Listening devices used by leak inspectors include listening rods or sticks and ground microphones, either of the mechanical or electronic type. These devices have sensitive mechanisms or materials, such as piezoelectric elements, that sense leak-induced sound or vibration. Modern electronic listening devices may feature signal amplifiers and noise filters that make the leak sound stand out. The effectiveness of listening devices in detecting leaks depends on the type and size of leaks; pipe material and diameter; interfering noise from road traffic and water draw; listening skill and experience of leak inspectors; the characteristics of the listening devices themselves.

Pipe material and diameter have a significant effect on the “loudness” and “travel distance” of leak sounds. For example, leak sound travels farthest in metal pipes but is attenuated (or dampened) greatly in plastic ones. The higher the frequency (i.e., tone or pitch) of the leak sound the more it is dampened with distance. Also, the larger the diameter of the pipe the thicker the pipe wall and subsequently the less responsive it becomes to vibration or sound generated by leaks, especially at high frequencies. Concrete and asbestos-cement pipes are almost as rigid as metallic ones but have much thicker walls and higher damping. Subsequently, they have lower pitched leak sounds.

As can be seen from the equal loudness contours of human hearing in Figure 1, the lower the frequency of sound below roughly 500 Hz, the less the sensitivity of human hearing. The lower the sound level the more pronounced this effect. In other words, sensitivity to sound decreases most sharply with decrease in the frequency of sound at the threshold of hearing. For instance, sensitivity of human hearing to sound at 500 Hz is about 10 and 100 times higher than at 100 and 50 Hz, respectively.

Subsequently, leak sounds in plastic pipes; concrete and asbestos-cement pipes; and large diameter pipes (> 300 mm ∅) which are dominated by low frequencies are hard to detect by listening surveys unless listening points are very close to leak locations. For example, listening devices were not effective for detecting a simulated leak in a 152 mm ∅ PVC pipe unless they were attached to access points that were roughly within 5 metres from the leak (Hunaidi et al., 2000). As a result, general surveys are unlikely to be effective in detecting leaks in non-metallic and large-diameter pipes. Also, detailed listening surveys may miss some leaks if the density of service connections is low.

The characteristics of leak sounds vary with leak type and size. Cracks and corrosion holes in pipe walls induce louder and higher-pitched leak sounds than leaks in joints or valves, and hence they are easier to detect by listening surveys. Generally, the larger the leak the louder the sound it generates, but this may not be true for very large leaks, as water flow becomes laminar or smooth. The higher the pipe pressure, the more turbulent is the leak flow and hence the louder and higher-pitched the generated sound. It is difficult to detect leaks in pipes having pressures less than ~15 psi.

There is significant variation in the sensitivity, frequency range, and signal-conditioning and processing features of different listening devices. The more sensitive the leak sensors and the higher the signal-to-noise ratio of the equipment (i.e., the lower the electronic noise of the equipment) the farther and smaller the leaks that can be detected. Some listening equipment has signal-conditioning functions such as filters and amplifiers to make leak sound stand out. Filters can be used to remove interfering noise occurring outside the predominant frequency range of leak signals. Amplifiers can be used to improve signal-to-noise ratio and make weak leak signals audible. Experienced inspectors with good hearing are able to discern weak leak signals and differentiate between interfering sound sources and actual leak sounds.

A general listening survey conducted in this study in the asbestos cement pipe DMA in Regina using an electronic listening device (Model LD-7 manufactured by Fuji-Tecom, Japan) at fire hydrants detected hydrant leaks only. The survey did not succeed in detecting other leaks that contributed to a sizable 393 L/minute water loss. This included a large 190 L/minute leak from to a circumferential crack in a 6-inch ∅ AC pipe, which was ~95 metres from the nearest fire hydrant. This large leak could not be heard on adjacent curbstops, although the noise propagation path did not include any PVC pipe splices as was confirmed from City pipe plans. It could only be heard on the curbstop of a service connection that was 10 to 20 cm away. It seems possible that similar large leaks in non-metallic pipes, if not sufficiently close to service connections, may be missed by detailed listening surveys.

Also, a general listening survey conducted in this study in the Meadowlands area of Ottawa did not succeed in detecting a large ~600 L/minute leak from a full circumferential crack in an 8-inch diameter cast iron pipe. The survey was conducted by experienced leak inspectors by listening at fire hydrants using an electronic listening device (Aqua-Scope from Health Consultants Limited). The large leak was ~70 metres away from the nearest fire hydrant. It could not be verified if there were PVC pipe splices between the leak and this fire hydrant.

LEAK NOISE MAPPING

Leak noise mapping is an enhanced form of general listening surveys that was developed at the Halifax Regional Water Commission after the utility did not realize desired reductions in leakage levels using general surveys (Brothers, 2001 and 2007). The noise mapping technique follows the usual practice of listening to leak sound at easily accessible contact points with water pipes. In Canada, fire hydrants are generally 150 metres apart and hence provide convenient contact points. Sounding is performed using electronic listening devices that are equipped with a sound level display, either analog or digital.

To minimize interference from other noise sources, especially in areas having busy road traffic and (or) high water use, surveys for leak noise mapping are performed at night, normally between 10:00 PM and 6:00 AM. It is very important to ensure that all acoustic listening devices are properly calibrated and procedures for attaching acoustic sensors are strictly followed by all inspectors to ensure consistency of measured leak sound levels.

Leak inspectors listen to leak sounds and document sound levels together with other standard descriptors such as pump or other mechanical noise or high-pitch noise (normally corresponding to fire hydrant leaks). Sound levels are input into a geographic information system (GIS) of the pipe network to produce coloured contour maps of leak sound levels. These maps enable leak managers to easily compare the latest sound levels with those previously collected to identify zones where more detailed surveys are needed. The maps also provide a visual view of leak noise patterns, which can help in identifying the presence of multiple leaks in a particular zone. Subsequently, leak managers can effectively plan the work of leak inspectors for validation and pinpointing of suspected leaks. Zones with validated or confirmed leaks are re-inspected after leak repair and leak noise is re-mapped to detect remaining leaks. The process is repeated until no leaks remain.

The noise mapping technique can be applied to metallic pipes but like other listening techniques cannot be effectively applied to plastic and other non-metallic pipe due to their significant noise dampening. Also, the technique may not be effective when applied to zones of metallic pipes with a large number of repairs using plastic pipe splices and/or plastic fire hydrant leads.

Brothers (2001) reported that a general listening survey and leak noise mapping of the 1100 km pipe network in Halifax that were conducted in series between early April and early June 2000 revealed a total of 32 and 216 leaks, respectively. The superior performance of the leak noise mapping technique in this particular case was attributed to the following factors Brothers (2001):

- Requirement for documentation of leak noise by inspectors;
- Listening at night;
- Visualization of leak noise patterns using contour maps of noise level; readings (i.e., less reliance on the skill and experience of leak inspectors);

- Control and planning by leak managers of leak validation and pinpointing activities; and
- Re-mapping of zones after leak repairs until no leaks remained.

LEAK NOISE CORRELATION

Many of the problems encountered by leak listening surveys can be overcome by correlator-based leak surveys. Detection of leaks by correlation of their sound is less susceptible to interfering noise from road traffic and water draw and subsequently correlation surveys can be carried out during the day, eliminating the need for night work and the safety hazards that it usually involves. Water draw may interfere with leak correlation if it is continuous and close to the leak. Short plastic pipe splices used for repairing metallic pipes generally do not affect correlation surveys. Leaks in plastic, asbestos-cement, concrete and large-diameter metallic pipes are detected by correlation surveys employing hydrophone sensors. Finally correlation surveys are not dependent on listening skills of leak inspectors.

Leak noise correlators are processor-based portable equipment that can be can be of the standalone type or implemented in software for common notebook PCs. The correlation method involves measurement of leak noise (either sound or vibration) at two locations on a pipe section using either accelerometers or hydrophone (vibration sensors or underwater microphones, respectively). Leak noise signals are transmitted wirelessly to the correlator, which then determines the position of the leak based on the time shift of the maximum correlation between the two leak signals, propagation velocity of sound in the pipe, and the distance between sensing points. The distance between sensors may be roughly read from pipe network maps when the correlator is used in survey mode but must be carefully measured on site when the correlator is used in pinpointing mode. Propagation velocities for various pipe types and sizes are programmed in most correlators, but they should be measured on site to improve pinpointing accuracy, especially for non-metallic pipes.

Two leak inspectors are needed to conduct a leak correlation survey. The route of the correlation survey is planned carefully in advance. Selected contact points, typically fire hydrants, are marked and numbered in sequence on two pipe network maps (one for each inspector), together with approximate distance and pipe type and size between them. One inspector attaches a first sensor and the other attaches a second sensor and runs the correlator. If a leak is detected, its approximate location is marked on pipe maps for later pinpointing. Inspectors communicate via radios to move their sensors between fire hydrants alternately or simultaneously in a frog-leap fashion.

A 2-person team skilled in the use of correlation can survey ~3 km/day (~15 minutes per correlation) for pipes that require the use of hydrophones, e.g., PVC pipes, and ~9 km/day (~5 minutes per correlation) for pipes on which accelerometers can be used, e.g., ductile and cast iron pipes. Km per day rates are based on the assumption of average hydrant spacing of 150 metres and an 8-hour workday of which 5 hours are spent on actual correlations, 2 hours on planning and 1 hour for travel to and from sites. Minutes per correlation rates were achieved in this study for surveys in small residential areas and involved correlation of consecutive fire hydrant pairs. Rates should double for surveys that involve the correlation of only alternate fire hydrant pairs. However, correlation of alternate fire hydrant pairs may miss some small out-of-bracket leaks because of the longer distance traveled by their sound.

Correlation based surveys conducted by experienced city staff in the Meadowlands' cast iron DMA in Ottawa did not succeed in detecting a large ~600 L/minute leak that a general listening survey missed. This is unusual and could be due to equipment problems (Hunaidi et al., 2000). NRC staff and city staff, using acceleration sensors at fire hydrants 193 metres apart, successfully correlated the missed leak with NRC's correlator.

In Regina, an initial correlation by NRC staff using hydrophones at fire hydrants ~216 metres apart did not detect a large ~190 L/minute leak in a 6-inch diameter asbestos cement pipe the Whitmore/Hillsdale DMA. This leak was also missed by an earlier general listening survey by city staff. However, the leak was successfully correlated at a later time. It was demonstrated that in the initial correlation, the leak was masked by a high-amplitude low-frequency out-of-bracket noise, most likely created by draw by a water softner(s), which are widely used in Regina.

LEAK NOISE LOGGING

Leak detection using noise loggers is a relatively recent alternative to periodic acoustic listening or correlation surveys. Noise loggers are compact units composed of a vibration sensor (or hydrophone), a programmable data logging module, and a communication module. They are deployed during the day in groups of six or more at underground valves, 200 to 500 metres apart.

The loggers are programmed to collect pipe noise levels at certain intervals during the quietest time of the night, normally between 2 and 4 AM. They are left in place overnight and collected the following day. Noise data is downloaded to a personal computer before the loggers are deployed at the next location. The data is analyzed using proprietary algorithms, mainly statistical frequency analysis of noise levels, to detect the presence of a dominant leak noise level.

Recent models of noise loggers can be deployed permanently. For these models, leak noise is measured nightly and processed using on-board processors and the result is stored in memory. The loggers wirelessly transmit alerts for detected leaks to a roaming receiver or a fixed one. A recent development also is to attach noise loggers to service pipes adjacent to customer water meters in order to exploit existing AMR communication infrastructure to wirelessly transmit leak alerts.

Other recent models known as “correlating noise loggers” can be programmed to record a sample of pipe noise at any time of the day or night. The noise record is stored in the logger’s memory and is later downloaded wirelessly to a PC via a roaming or fixed receiver. A correlation between noise records from logger pairs is performed using proprietary software to pinpoint suspected leaks.

Frequent claims are made about the superior performance and cost effectiveness of acoustic noise loggers in comparison to traditional acoustic leak detection surveys. However, such claims remain unconfirmed – actually, documented field trials by three water utilities in the United States, the United Kingdom and Canada do not support such claims. These trials, which are summarized below, involved head-to-head evaluation of noise loggers with acoustic listening surveys.

Albuquerque Bernalillo County field trial

The Water Utility Authority of Albuquerque Bernalillo County in New Mexico assisted by the New Mexico Environmental Finance Center conducted a major study to compare the performance of noise loggers and acoustic listening surveys (NMEFC, 2006a, 2006b & 2007). The make of the loggers was Permalog®, manufactured by Palmer Environmental Ltd. in the U.K. and distributed in the U.S. by Fluid Conservation Systems (FCS). FCS prepared, deployed and patrolled the loggers under contract with the county. The county also contracted an independent leak detection firm, Hughes Supply Inc., to conduct listening surveys. Hughes survey crew used acoustic listening equipment to listen for leaks at fire hydrants, valves, meter, service lines etc., at predetermined intervals. Crews from both firms used the AccuCorr® leak noise correlator, manufactured by Palmer Environmental Ltd. in the U.K., to pinpoint the location of suspected leaks.

Both firms utilized their respective leak detection methods during the same time period on the same section of water pipes within four grid zones of the county, each having an area of 2.6 square km (1 square mile). Grid zones were selected to represent different pipe types in different areas of the county. One zone had a high percentage of PVC pipes (41%). The other three zones had a mix of cast and ductile iron (CI/DI), asbestos cement (AC), concrete cylinder (CCYL), steel and galvanized steel (STL/GSP) and PVC pipes that varied from zone to zone but was dominated by CI/DI pipes. All zones had almost the same length of pipe which had contact points that were at least ~100 meters apart. Altogether, ~158 km of pipes were tested, which on average consisted of 72% CI/DI, 18% PVC, 9% AC/CCYL and 2% STL/GSP. About ~400 noise loggers (~100 per zone) were deployed at valves ~400 metres apart, on average.

Noise loggers detected 13 leaks, while 41 leaks were detected by listening surveys. There was very little correlation between leaks detected by the two methods, even for leaks on distribution pipes. Out of the total 54 detected leaks, the two methods only had 3 leaks in common. Only 3 of the 13 leaks detected by noise loggers and 25 of the 41 detected by listening surveys were confirmed by utility staff. For noise loggers, the majority of false leaks were at distribution mains (6) while the majority of false leaks detected by listening surveys were at fire hydrants (11) and meters (5).

It is unlikely that experienced leak inspectors confused the distinctive high-pitched leak noise at fire hydrants. The large number of false leaks detected by listening surveys at hydrants and meters could be a matter of definition, e.g., reported fire leaks did not require repair but proper closing of hydrant valves. The large number of false leaks detected by noise loggers could be an indication that the devices were triggered by noise from sources other than leaks, i.e., the devices did not function properly. Another possibility is that leaks were improperly correlated or somehow utility crews failed to locate the reported leaks. The latter is unlikely to be a major factor because the majority of leaks detected by noise loggers were on distribution mains and none of them could be confirmed (i.e., unlike for fire hydrants, repair is not a matter of definition.) Also, noise loggers failed to detect any of the leaks detected by listening surveys that were confirmed by utility staff.

On average it took ~4.25 minute/km to patrol noise loggers and ~38.5 minute/km for detailed listening surveys. It took FCS and Hughes crews ~25.6 and 29.2 minutes, respectively, to pinpoint a suspected leak. Assuming that noise loggers have a life span of 36 months and that they are patrolled once a month, their cost was reported reported to be ~$187/km, compared to $172/km for listening surveys.

The expected life of the loggers is claimed to be 8 to 10 years. However, following the head-to-head evaluation and 15 months after the Permalog® noise loggers were deployed, county staff started experiencing problems with some of the loggers. They were not responding during routine patrols, may be as a result of malfunctioning batteries, incorrect antennae or incorrect installation. Las Vegas Valley Water District reported similar difficulties with probably half of their 8000 noise loggers (Saunders, 2009).

Bristol Water field trial

Bristol Water Plc in the U.K. undertook a two-stage study to evaluate the performance and operational benefits of several types of noise loggers in comparison with listening surveys by skilled inspectors (van der Kleij and Stephenson, 2002). Three types of loggers were evaluated: (i) Non-correlating loggers with RF wireless interrogation capability; (ii) Non-correlating loggers without strobe light interrogation capability; and (iii) correlating loggers without wireless interrogation capability.

In the first stage of the study, the three logger types were evaluated in a trial area against the performance of general listening surveys at valves and fire hydrants. The trial area was surveyed for leaks prior to the deployment of each logger type. Loggers were then programmed, deployed, left in place for several nights, and then interrogated for results. Detected leaks were followed up by leakage inspectors and pinpointed using usual techniques. The three logger types found the most significant leaks within the trial area but missed some leaks found by general listening surveys. When deployed in lift-and-shift mode (i.e., temporarily), none of the three logger types could economically outperform general listening surveys, even without considering initial capital cost. Loggers were also economically unappealing when deployed on permanent basis, having a payback period of at least 26 years.

In the second stage of the study, non-correlating loggers without wireless interrogation capability were evaluated in more detail in comparison to listening surveys. Loggers were deployed in lift-and-shift mode and both general and detailed listening surveys were carried out. Tests were conducted in several areas having different pipe materials, traffic densities and consumption patterns, and included areas that traditionally required night listening surveys. The numbers of leaks found by noise loggers and by general listening surveys were similar; however, the loggers failed to detect approximately 40% of leaks found by detailed listening surveys. The majority of leaks missed by the loggers were the typically smaller ones on service pipes, valves and fire hydrants. Time wise, detailed listening surveys were as efficient as the use of noise loggers. However, general listening surveys were three times faster than the use of noise loggers.

It was recognized that noise loggers could offer a more effective alternative to listening surveys in areas with significant interfering noise from heavy road traffic and water use during the day, as well as areas having variable night consumption, e.g., entertainment districts. As such, Bristol Water considers noise loggers as a “specialist leakage investigation tool”, but they are not suitable as an across the board replacement for listening surveys.

HRWC field trial

Halifax Regional Water Commission (HRWC) in Halifax, Nova Scotia, evaluated a set of 8 correlating noise loggers in selected areas of their water pipe network (Fanner et al., 2007). These areas had been acoustically surveyed and noise mapped by HRWC staff utilizing fire hydrants as contact points. Some of the selected areas had leaks while others had none detected by HRWC staff.

Noise loggers were programmed to listen for several intervals during the night, deployed during the day at system valves, left in place overnight, retrieved and analysed the following day. Considerable time was spent removing dirt and debris from valve chambers and cleaning the surface of valve nuts where loggers were attached via strong built-in magnets. The loggers were able to detect the leaks previously detected by HRWC leak detection teams; however, they did not detect any other leaks.

The 8 correlating loggers were also deployed at valves around known leaks in two other pipe sections. The first section was an 8-inch ductile iron pipe having a very small leak (believed to be less than 4 litres/minute) that was barely detectable at adjacent contact points by leak inspectors. The second section was an 8-inch PVC pipe having a simulated leak that was easily detectable by leak inspectors using an amplified ground mic on the ground surface above the pipe. In the case of the ductile iron pipe, only the two loggers closest to the leak were able to detect and correlate it. The loggers failed to identify the simulated leak in the PVC pipe.

The cost of the 8 correlating loggers including software was $20,000, which HRWC considered to be expensive for permanent deployment across their pipe network. It was recognized that noise loggers could return false alarms as a result of sounds induced by control valves, and electronic noise at the electricity mains frequency. However, it was noted that the loggers would be a valuable tool in areas that require listening surveys during the night, e.g., downtown core and industrial/commercial areas. It was also recognized that the loggers were relatively simple to use and required no special skills.

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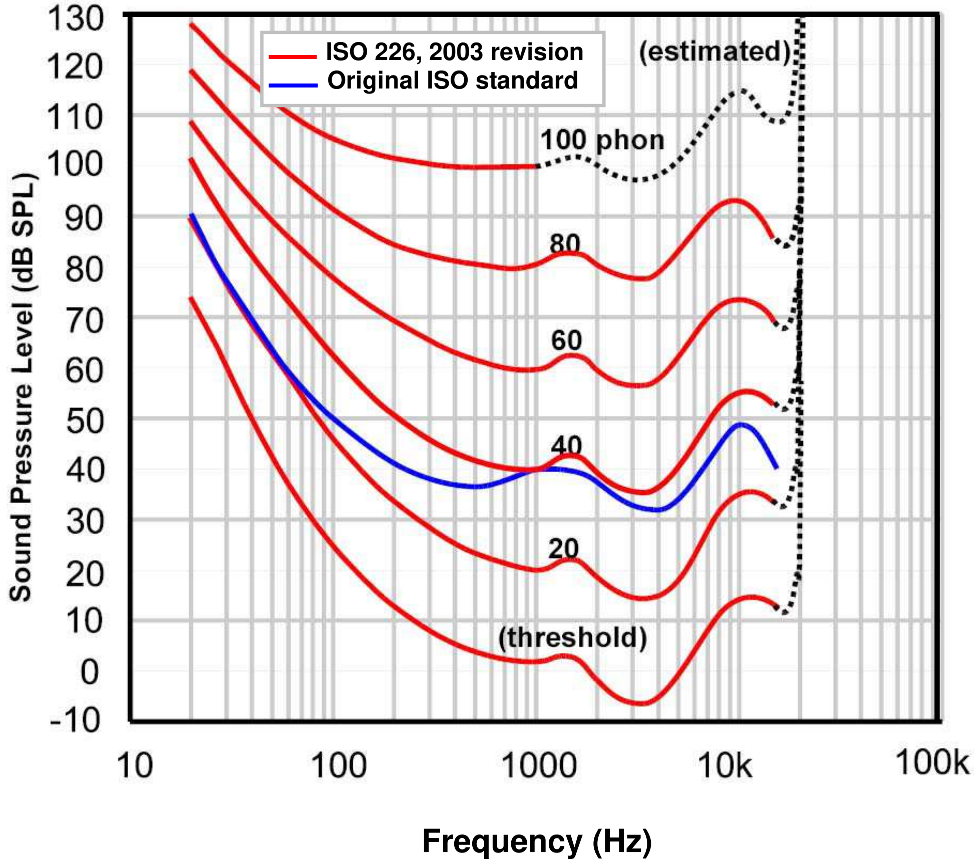

Figure 1: Equal-loudness contours of human hearing (Note: This graph has been released into the public domain by its author, Lindosland, at the English Wikipedia project)

APPENDIX B

A GIS-BASED SOFTWARE SYSTEM FOR IWA'S WATER BALANCE METHOD

(Mahmoud Halfawy and Osama Hunaidi)

INTODUCTION

A number of “best practice” (BP) documents that define standardized water balance analysis procedures and guidelines have been developed. Most notable are the American Water Works Association (AWWA) and the International Water Association (IWA) manuals. AWWA guidelines for performing a water audit were first published in 1990 in their Manual M36 titled “Water Audits and Leak Detection,” and updated in 1999. IWA published guidelines for conducting a water balance in the year 2000 in their manual of best practice titled “Performance Indicators for Water Supply Services” and in a Blue Pages paper titled “Losses from Water Supply Systems: Standard Terminology and Recommended Performance Measures.” Both manuals recommended procedures for quantifying water loss, computing the amount of recoverable leakage, evaluating economic aspects of different leakage management approaches, and calculating a set of performance indicators (PI). In 2003, the AWWA started an effort to harmonize its manual M36 with the IWA water balance methodology.

The IWA BP has been adopted by water utilities around the world. However, utilities implementing the IWA water balance analysis procedure face significant challenges to collect and manage the required data. Successful implementation of water balance analysis procedures would depend, to large extent, on the ability to efficiently collect and integrate data from various sources including metered and un-metered consumption, water supply records, and the physical characteristics and asset inventory of the water network (e.g., length and number of service connections). The majority of existing water balance analysis software require users to manually collect the data from these multiple sources (e.g., pumping and billing records), and input the data into a form or spreadsheet, a process known to be time-consuming, inefficient, and prone to errors.

This appendix presents the implementation of a GIS-based software system for water loss analysis based on the IWA BP. The software integrates data from water network asset inventory, water supply supervisory control and data acquisition

(SCADA) records, automated meter readings (AMR), and other water use data to automatically generate water balance reports, quantify water loss and recoverable leakage, and calculate water loss key PI. The software was developed and validated using a sample data set from the City of Regina.

DEVELOPMENTS IN WATER SUPPLY AND CONSUMPTION DATA COLLECTION SYSTEMS

Availability of reliable water supply and consumption records is a prerequisite for implementing effective leakage management strategies. Recent advances in flow metering and automated meter reading (AMR) systems offer utilities new capabilities in collecting accurate and timely supply and consumption data. Many utilities are now using electromagnetic and non-intrusive ultrasonic flow meters, equipped with efficient data logging and telemetry devices. Many utilities have upgraded their SCADA systems to include more monitoring and control capabilities for treatment plants, pumping stations, and reservoirs, enabling the collection of accurate supply data in almost real-time.

Rapid advances in automated meter reading systems during the past decade have also offered utilities new opportunities to obtain accurate, reliable, and timely consumption data, thus ensuring accurate billing information and significantly reducing operating costs. Over a relatively short period of time, AMR systems have evolved from walk-by, to drive-by, to fixed network. Most of today's AMR systems are drive-by, where receivers are mounted on a vehicle that passes by customer properties to collect readings from transmitters attached to meters, or fixed networks, where a permanent communication infrastructure is installed to transmit meter readings on a regular or on-demand basis to a base station. Fixed network systems are capable of providing real-time consumption data. Moreover, fixed network AMR systems are more power-efficient than mobile systems, since readings will have to be transmitted only a few times as opposed to continuous transmission. Fixed network AMR systems typically have higher capital costs, and less operating costs than mobile systems. However, with the decreasing cost and increasing range of modern wireless communication devices and the increasing operating and labour costs, the fixed network systems are becoming more cost-effective on the long run, causing many utilities to implement fixed network systems.

Metering data have traditionally been used, almost exclusively, for billing purposes. Even for utilities that already have an AMR system in place, the use of AMR data has yet to be extended beyond billing. These data can be invaluable for a wide range of applications such as water balance analysis, accurate assessment of per capita consumption, demand management, preparing accurate hydraulic models with accurate node demand levels, etc. The software described in this paper attempts to leverage the use of supply and consumption data, and integrate these data with other inventory and operational data to provide utility managers and operators with a practical tool to perform water balance analysis in as much automated way as possible.

THE IWA WATER BALANCE ANALYSIS PROCEDURE

The IWA procedure divides water losses into real (or physical) losses and apparent (or non-physical) losses. Real losses are caused by pipe leaks and breaks before they are repaired, and by reservoir leaks and overflows, while apparent losses are caused by customer meter under-registration, accounting errors, and unauthorized water use. Identifying the real and apparent components of lost water facilitates the calculation of the true financial value of the loss, and thus justifies proactive leak management programs. Figure 1 shows the IWA components for water balance analysis.

The IWA water balance analysis procedure requires the measurement or estimation of the following components:

a. Corrected water input to the system;

b. Billed uncorrected metered use (i.e., including customer meter errors);

c. Billed un-metered use;

d. Unbilled metered use;

e. Unbilled un-metered use;

f. Unauthorized water use; and

g. Customer meter errors.

The analysis procedure computes the following components:

a. Revenue water (or authorized billed use) equal to component (b) plus (c)

b. Non-revenue water equal to component (a) minus (h)

c. Unbilled authorized use equal to component (d) plus (e)

d. Authorized use equal component (h) plus (j)

e. Water losses equal to component (a) minus (k)

f. Apparent losses (including customer meter errors, accounting error, and unauthorized use) equal to component (f) plus (g)

g. Real losses equal to component (l) minus (m)

The IWA water balance method does not calculate the volume of recoverable leakage as a simple percentage of real losses. Instead, it provides an equation to calculate a value representing the low-level annual real loss that is technically achievable at current operating pressure, if there were no economic or financial constraints. This value, known as Unavoidable Annual Real Losses (UARL), is calculated using equation [1], and takes into account the following local characteristics of the water system: (i) length of transmission and distribution mains in kilometres (Lm), (ii) number of service connections (Nc), (iii) average length of service connection pipes in metres (L_p) prior to customer meter or curb stop, and (iv) average operating pressure in metres (P).

[1] $\text{UARL (litres per service connection per day)} = (18 \times L_m / N_c + 0.7 + 0.025\, L_p) \times P$

Recoverable leakage is assumed equal to real losses minus UARL. The UARL equation was derived by following the burst and background estimate (BABE) component-based approach, which splits real losses into 3 components: (i) background losses from acoustically undetectable leaks (with flow rates less than 500 litre per hour), (ii) losses from breaks reported by the public (these are large leaks that surface or cause visible damage), and (iii) losses from unreported breaks detected as a result of active leakage control (utilizing district metering and acoustic surveys). Background losses are estimated from minimum night flows of district meter areas (DMAs) after all detectable leaks have been located and repaired. Losses from reported and unreported breaks are estimated from system records for frequencies, average flow rates and durations of breaks (loss = break frequency x flow rate x duration).

The UARL equation was primarily based on data from U.K. and German systems for background losses, break frequencies, unreported break durations, and flow rates (Lambert et al. 1999). The relationship between leakage flow rate and operating pressure is assumed to be linear, which may underestimate losses in systems consisting mainly of plastic pipes and overestimate them in systems with mainly metallic pipes. The 500 litre per hour set for acoustically undetectable leaks is probably outdated. Recent acoustic leak detection equipment can detect leaks with flow rates much smaller than this threshold. Therefore, values obtained from the UARL equation may have to be adjusted to account for a number of local factors, which are not considered in the equation. Example of these factors include:

- Physical condition of the infrastructure;
- *Pipe material and size*. North American systems with pipes larger than those in Europe are expected to have different background losses, break frequencies and flow rates;
- *Soil type and pipe burial depth*. For example, pipes laid in sandy soils may be less prone to differential movement, and are generally expected to have less breaks and water loss than those in clayey soil. Also, pipes buried deeper, such as in northern countries, are less susceptible to the effect of

traffic loads and vibration and may have smaller breakage rates due to these effects;

- *Climate conditions*. Pipes in northern climates are subject to higher breakage rates due the large seasonal variation in water temperature and frost loads; and
- *Adopted acoustic leak detection procedures*. For example, a correlator-based survey may detect a larger number of leaks than a listening survey.

CALCULATION OF IWA PERFORMANCE INDICATORS

Water loss performance indicators (PIs) play an important role for leakage benchmarking and reporting, and for supporting operational and renewal planning decisions. The IWA manual recommends a set of seven PI for water loss management in the following three domains: (i) water resources (environmental), (ii) financial (revenue related), and (iii) operational (technical aspects of leakage control). An understanding of the significance and inter-relationships of these PIs is important for their effective use. Since it may be too demanding for utilities to calculate all recommended PIs, the IWA manual recommends a step-by-step implementation, and for this purpose PIs are ranked according to three priority levels: Level 1 indicators are high priority and they provide management with an overview of the efficiency and effectiveness of the utility; Level 2 indicators provide intermediate details for better insight; and Level 3 indicators provide the greatest amount of specific details.

The PI for efficiency of use of water resources is calculated as the percentage of real losses in terms of system input. This PI was found to yield a misleading indicator, e.g., in systems with high levels of unauthorized consumption (Liemberger 2002). The IWA manual considered this PI as unsuitable for assessing the efficiency of the management of the distribution system.

The PI for operational efficiency is measured as the annual volume of losses per service connection per year. This is because, as shown by experience, the major part of unavoidable and avoidable water losses are attributed to service connection leaks, except for systems with less than 20 service connections per km (Lambert and Hirner, 2000). Therefore, expressing this PI in units of service connections provides a more suitable measure than units of km of pipes, for the purpose of comparing the performance of different utilities.

The PI of current annual real losses (CARL) may not be readily apparent without consideration of the UARL value. The PI, termed Infrastructure Leakage Index (ILI), is defined as the ratio of CARL to UARL, can provide a good indicator to the level of real losses. ILI was found to be more suitable for benchmarking the leakage control performance of different utilities. In general, well-managed systems, with infrastructure in good condition, will have ILI value close to 1, while old systems with deficient infrastructure and less active leakage control will have higher values. Therefore, a high ILI value would indicate a high potential for reducing water losses by increased leakage control activities, improved response time and quality of repairs, and proactive infrastructure rehabilitation. However, locating and repairing all detectable leaks in the shortest time may not necessarily lead to an ILI that is equal 1. The minimum achievable ILI is, to a great extent, dictated by the infrastructure condition.

Also, an ILI equal to 1 is not necessarily ideal for a particular utility because it may not be economic to achieve, even for systems with infrastructure in good condition. Therefore, comparison of the ILI of different utilities may be a misleading indicator for leakage control purposes if economic factors are not taken into account. For example, two utilities with equally active leakage control programs may have different ILIs (e.g., 2 versus 4). The utility with the lower ILI may be incorrectly perceived as operating more effective leakage control program, although both have implemented similar programs but as dictated by economic leakage levels. Other factors that may lead to misinterpretation of a utility's ILI in comparison to those of other utilities is inaccuracies in UARL calculation caused by the local factors mentioned above and not taken into account by the UARL equation, e.g., pipe type and size, soil type, etc.

There have been strong debates in the water loss community on the use of ILI as a performance indicator. However, many specialists contend that despite its perceived limitations, ILI provides a valuable measure that “should be considered as indicative rather than totally rigorous” (Seago et al. 2005).

SOFTWARE TOOLS FOR WATER BALANCE ANALYSIS

During the past decade, several efforts have been made to develop software tools that assist utility managers and operators in implementing standard water balance analysis models. The vast majority of these software tools are based on the IWA BP for annual water balance analysis. Most notable among these tools are Aqualibre and Benchleak.

Aqualibre (Liemberger and McKenzie 2003) implements the IWA top-down approach and uses the IWA water balance form to estimate the amount of real losses from user-specified values for total system input, authorized billed consumption, and apparent losses estimate. If reported burst data are known, the system also calculates the real losses using the bottom-up BABE approach (Lambert 1994) using estimates for apparent losses and background leakage, and calculates the amount of unreported bursts that need to be identified via active leak detection programs. The estimated values for various components are revised until reasonable and consistent estimates for real losses, unreported bursts, and apparent losses can be established. Aqualibre allows users to define 95% confidence limits on input variables.

Benchleak (McKenzie et al. 2002) is an Excel spreadsheet tool that was developed by the South Africa Water Research Commission, and released in 2001. Benchleak enables leakage benchmarking between different utilities using the infrastructure leakage index (ILI) performance indicator, which is calculated as the ratio of the current annual real losses (CARL) to the unavoidable annual real losses (UARL). The Benchleak model has been adapted and enhanced in Australia (the Benchloss model) and New Zealand (the BenchlossNZ model) (McKenzie and

Seago 2005). Another IWA-based spreadsheet tool, called FastCalc, was recently developed (Wide Bay Water Corporation, 2008). The AWWA also developed and made freely available an Excel spreadsheet for water balance analysis. However, most of these tools offered the same fundamental function and implemented in a straightforward manner the equations proposed in the IWA manual. Seago et al (2005) noted that most water balance software tools supported the IWA water balance analysis, and their differences are mainly “cosmetic.”

A main difference between the software described in this paper and other tools is that it integrates directly with multiple data sources, thus eliminating the need to manually collect and re-input the data. It also integrates with a set of condition assessment and asset management tools (e.g., hydraulic modeling tool, historical leak data analysis tool, condition assessment tool, etc.). This integration is critical in developing and implementing an integrated proactive approach for managing water networks on both operational and strategic levels. The need to consider leakage management as an integral part of an overall asset management strategy for the entire water network has been emphasized by several researchers (e.g., Parker 2005).

SOFTWARE IMPLEMENTATION AND EXAMPLE APPLICATION

The water balance analysis software was implemented as part of an integrated GIS-based asset management system being developed in collaboration with the City of Regina. The integrated system aims to support various processes conducted by functional groups within a typical municipal water department. In addition to the water balance tool, several applications have been under development during the past three years. These applications support such processes as inventory data analysis and query, water mains break data analysis, hydraulic modeling, deterioration modeling, risk assessment, selection of renewal technologies, and renewal planning based on a multi-objective optimization methodology. This section describes the implementation of the water balance tool, based on the IWA methodology described above.

The water balance software was implemented as an add-on to ESRI ArcGIS software using the ArcObjects class library (ESRI 2001). Unlike other water balance analysis tools, the proposed software significantly facilitates the data entry process by directly integrating with data sources, thus overcoming the limitations associated with manual data collection and entry. The software integrates data from GIS asset inventory, water supply SCADA records, AMR records, and other water use data, to generate water balance reports, quantify water loss and recoverable leakage, and calculate water loss key PI. The software implements a centralized GIS-based integrated data repository to support efficient access, integration, and management of these data sets (Halfawy and Figueroa 2006). Accessing the data repository through a unified GIS interface significantly enhances the ability to explore, access, query, and edit the data. The GIS interface also enables spatial characterization of the leak and break distribution across the water network. The use of an integrated data repository can achieve the following benefits:

- Enable water utilities to gather, manage, and analyze data more efficiently. It also facilitates data archiving for future analysis and audit trails.
- Leverage the use of GIS data by linking various lifecycle data to it.
- Enable data sharing and the interoperability of various software applications.
- Enable data reusability and eliminate redundancy and possible inconsistencies in collecting, validating, entering, and storing the data.
- Streamline various asset management processes by enabling efficient data flows among users and software systems supporting these processes.

The data repository assists in managing both spatial and non-spatial data of the water network. Spatial data describe the physical characteristics of transmission and distribution water mains, service connections, consumer meters, as well as data about hydrants, valves, fittings, pump stations, and reservoirs. Non-spatial (or lifecycle) data describe water main break data, hydraulic attributes, risk data, and maintenance operations and renewal plans. Water supply records are provided in a spreadsheet format for the two main pumping stations in the city. The data is exported and stored in a relational table with attributes for daily total production of each pump station. The supply data can be plotted for a particular pump station or for the total production (Fig. 2).

The City of Regina is using a drive-by AMR system, where readings are generally taken once or twice a month, however, on arbitrary days. The billed metered consumption data are stored in the billing system database. The records are exported from that database in the form of a comma-separated file, where each line includes a meter reading record. This file is used to populate the *MeterReadings* relational table in the repository. Each meter reading record has an attribute that indicates the reading status, which is used for billing purposes. Four states are used: IN, OUT, SKIP, and READ. The IN status indicates that a new customer moved into the property; the OUT status indicates that a customer moved out of the property; the SKIP status indicates a bad reading that should be ignored; and the READ status indicates a normal reading.

The software first examines the *MeterReadings* table to ensure the validity of the readings and reading states. Any anomalies will be identified and the user will be notified (e.g., if a meter indicates an unusual high, low, or missing reading). Then, the software calculates the average daily consumption for each meter, during the periods between consecutive readings. The calculation results are then used to populate database tables, which will be used for performing the water balance analysis and calculating the IWA performance indicators.

The software implements efficient data query and visualization functions to enhance an operator's ability to understand and assess water use patterns across the network at the level of zones or individual meters. The GIS meters layer is spatially joined with the zones layer, and subsequently, the average daily or monthly consumption can be easily determined for different zones or for the entire network (Fig. 3). Water use patterns over any period of time can be calculated and plotted for a particular meter, zone, or for the entire network. Meters are classified as residential, multi-residential, commercial, and irrigation. Daily consumption for each category of meters can also be displayed, for individual zones or for the entire network. It should be noted that water network in Regina is not divided to DMAs, and the entire network is primarily one large pressure zone. The current version of the software uses the city's different subdivisions to summarize water use patterns, and other statistics (e.g., main breaks). However, the software can be easily adapted to situations where DMAs or pressure zones are used.

Figure 4 shows a screenshot that displays the monthly water supply, billed consumption, and non-revenue water for the entire city between March and November 2005. The data used in this analysis included:

1. Water supply SCADA records for the North and Farrell pumping stations,
2. Records of the city's drive-by Automated Meter Reading (AMR) system for the period starting February 2005 to December 2005; and
3. GIS data of the city's water distribution and transmission network.

The city has a bi-monthly billing schedule under which meters are typically read once every two months following a routing schedule prepared by the billing department. Therefore, for the purpose of this audit, subsequent bimonthly AMR readings were used to calculate an average daily consumption for the period between readings dates. Total network-wide monthly consumption was calculated based on the average daily consumption. Monthly non-revenue water figures were as follows: 21.3% (March), 18.2% (April), 22.4% (May), 16% (June), 22.4% (July), 33% (August), 27.7% (September), 25% (October), and 26.4% (November). Average total non-revenue water for the nine-month period between March and November 2005 was found to be approximately 23.9%, i.e., (21704.5-16511.7)/21704.5 = 23.9%. The apparent losses component of non-revenue water in the city is believed to be negligible, as the majority of flow meters in the city have been recently replaced.

Hence, non-revenue water will have to be mostly attributed to real losses due to leakage.

The software implements the IWA methodology to calculate water loss performance indicators (e.g., UARL, CARL, ILI) (Figure 5). These calculations can be performed on annual basis or to cover arbitrary periods of time. A utility that employs a fixed network AMR system and real-time connection to supply flow meters can perform water balance analysis in almost real-time, and provide operators with an immediate feedback about leakage levels, which could prompt the implementation of a leak control strategy to reduce the water losses (e.g. by adjusting the pressure).

Calculation of water loss PIs involves the retrieval of water mains and service connections physical attributes from the data repository. The main attributes needed are the total length of transmission and distribution mains, the number and average length of service connections. The software performs a spatial join of the water main and the service connections layers along with the zone layer to identify the pipe segments and service connections that lie within a particular zone. A similar join is performed between the hydraulic model node layer and the zone layer to identify the network nodes that lie within each zone. Accordingly, the total length of water mains, number and average length of service connections, and the average operating pressure can be automatically calculated and used to compute the UARL value. The user also enters the values of metered unbilled water use (e.g., hydrant permits, unidirectional flushing), and estimates for any authorized unmetered water use (e.g., fire fighting, street sweeping, sewer jetting, and other operational use). The software allows users to specify +/- % error on key input variables (e.g., billed/unbilled metered/unmetered use) to consider 95% confidence limits.

In addition to the water balance analysis, the prototype software can play an important role in supporting other functions. For example, leakage levels for different zones can be used along with other water main condition indicators (e.g., break distribution) to prioritize maintenance, leak survey, inspection, and NDT activities by focusing on zones that experience high leakage and break rates. This prioritization would enable municipalities to optimize the allocation of limited resources by focusing on critical zones. Also, linking the water balance software with the hydraulic modeling tool would support activities such as performing what-if simulation and analysis scenarios, the DMA design process (e.g., selection of appropriate DMA size, selection of flow meters, etc.), devising efficient pressure management schemes, and selecting appropriate leakage management strategies. Also, the actual metered average daily consumption can be used to assign accurate node demands, instead of using approximate demand values.

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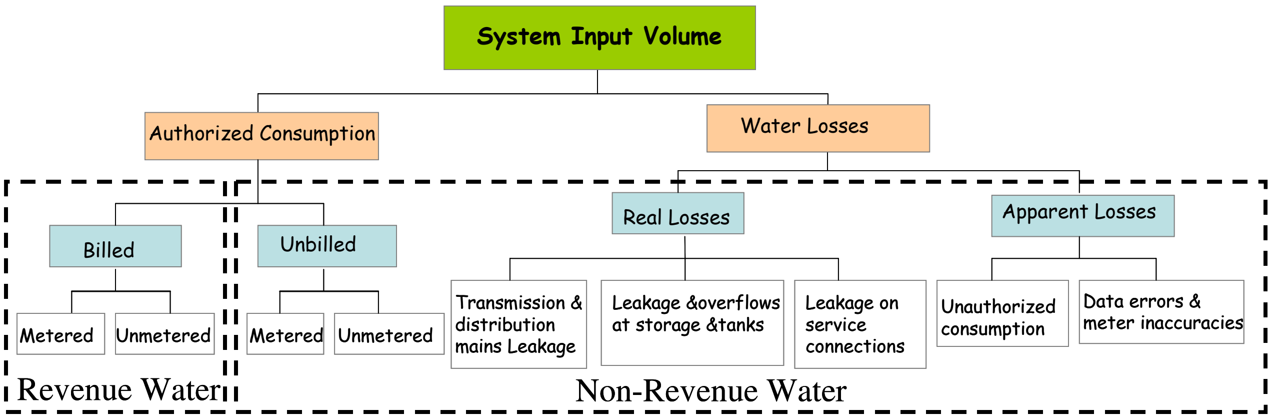


Figure 1: IWA Components for Water Balance Analysis

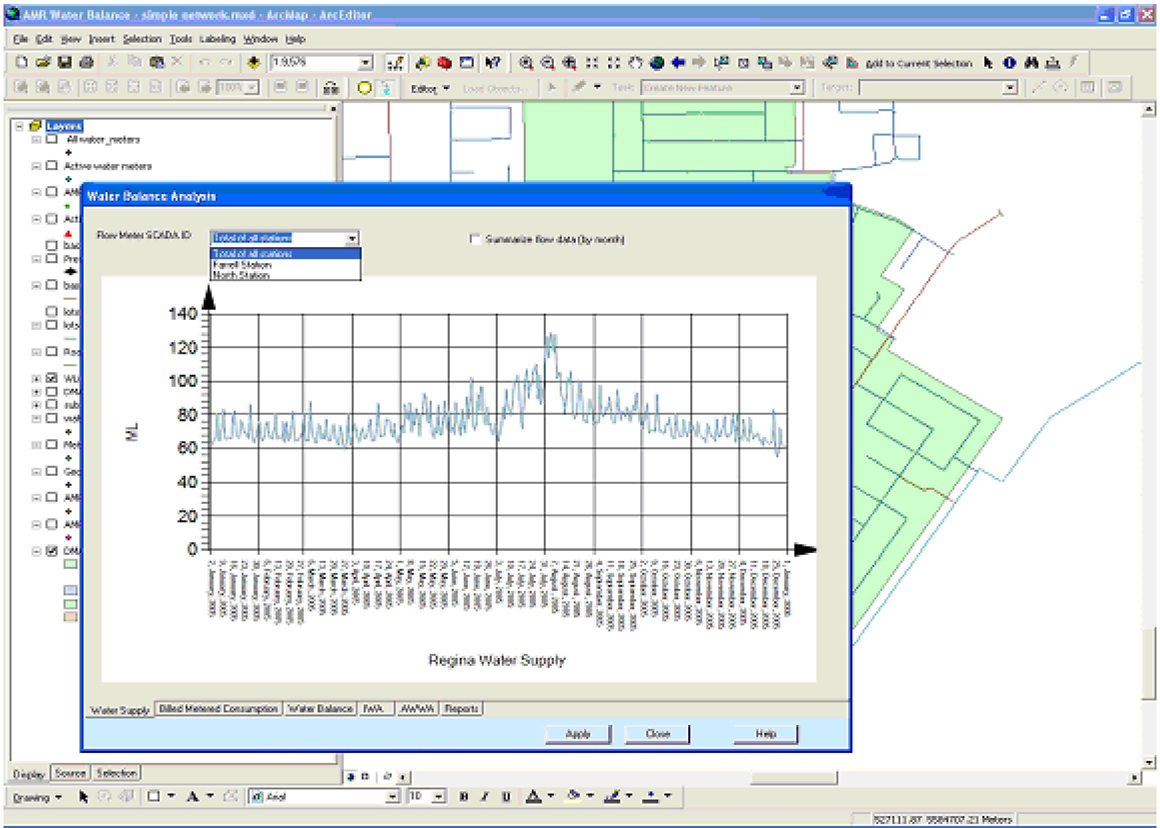


Figure 2: Daily water supply data from the SCADA records

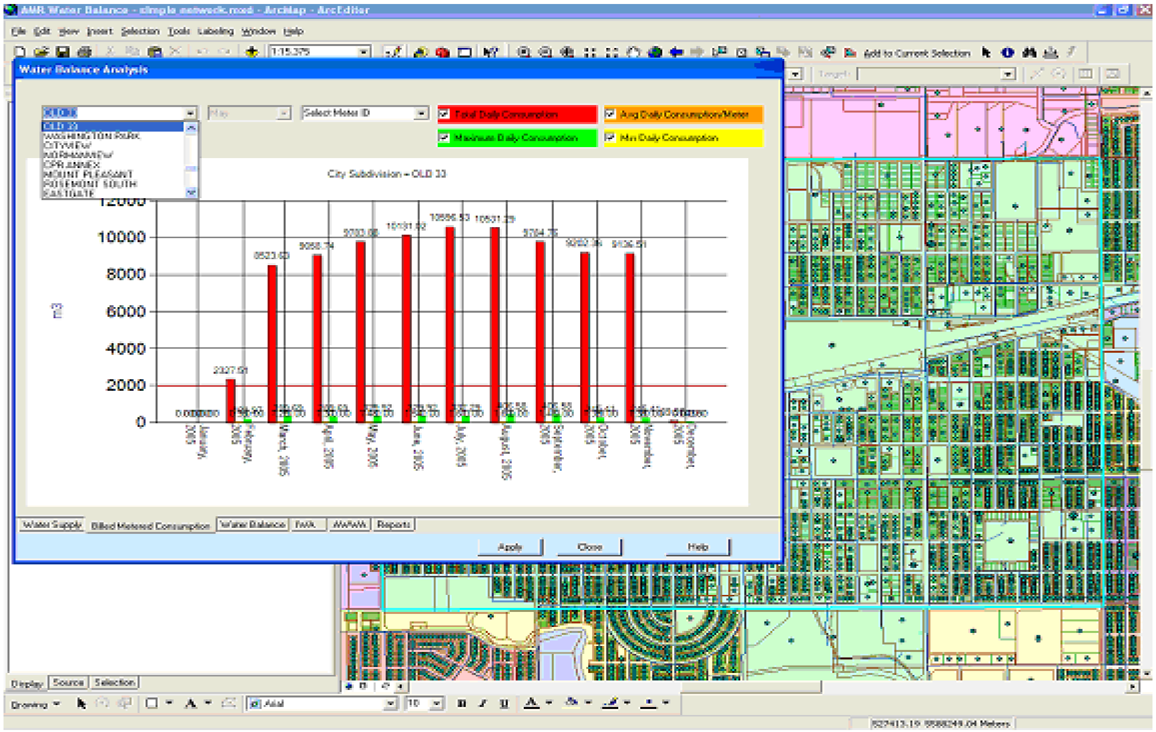


Figure 3: Monthly summary of billed consumption for a particular zone

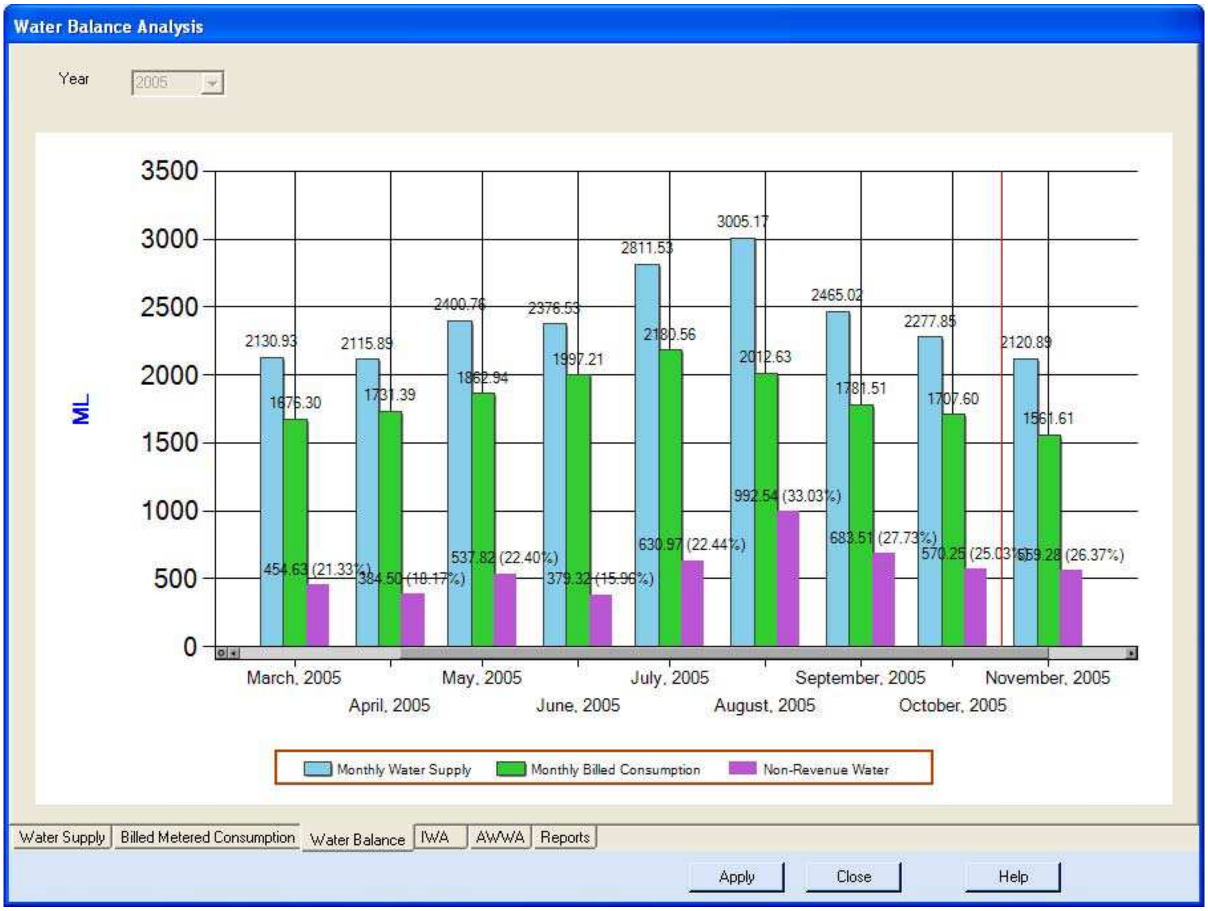


Figure 4: Monthly water supply, metered consumption and non-revenue water for Regina's water distribution network between March and November 2005

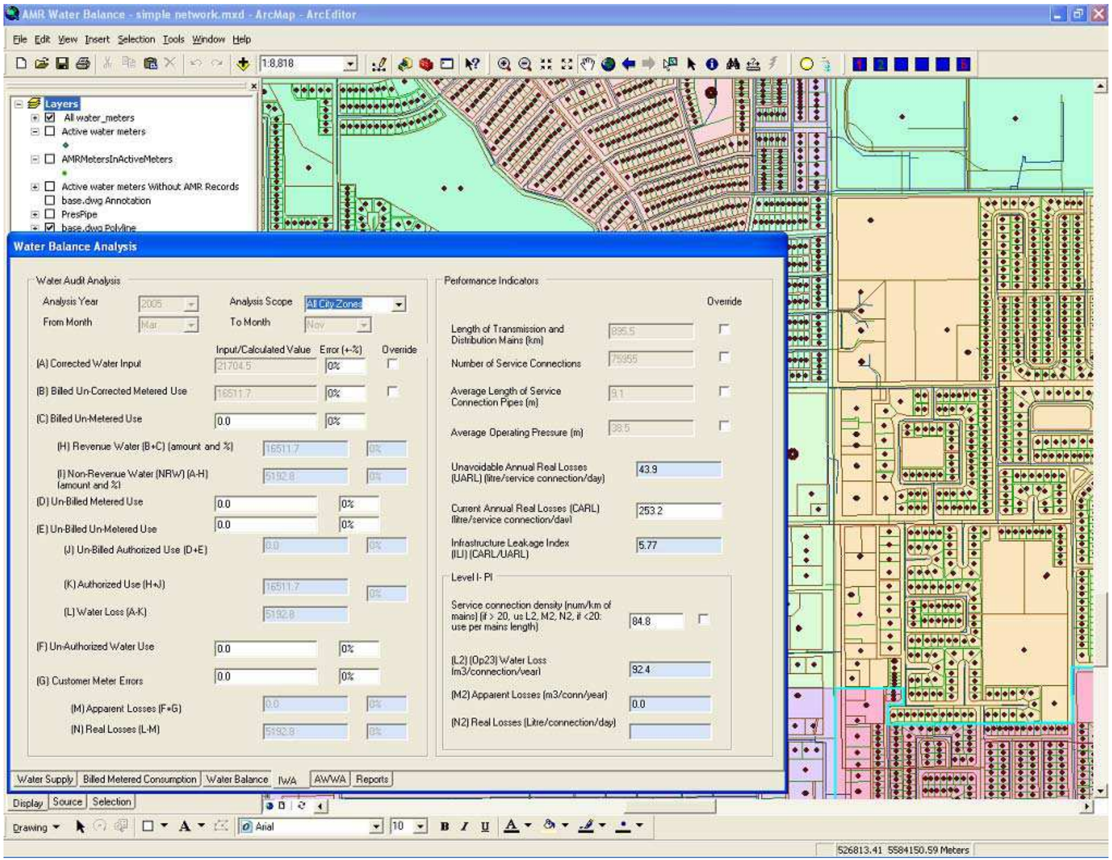


Figure 5: Sample calculation of the key PIs for the entire water network

APPENDIX C

CHARACTERISTICS OF DMAs

(provided by City of Ottawa)

Ductile iron DMA – Orleans

Pipe Count by Diameter

Type	Internal Diameter	Count	Kilometers	% by size
DI	152	43	8.44	30.1%
DI	203	31	3.00	21.7%
DI	305	55	4.99	38.5%
DI	406	14	2.00	9.8%
	Total	143	18.43	100.0%
PVC	203	11	1.38	68.8%
PVC	305	5	0.15	31.3%
	Total	16	1.53	100.0%
Undefined	Undefined services	35	1.78	100.0%

Summary Pipe Count by Material Type

Type	Count	Kilometers	% by length
DI	143	18.43	84.8%
PVC	16	1.53	7.0%
Undefined	35	1.78	8.2%
Total	194	21.74	100.0%

Land use by Parcel (with duplicates removed)

LAND_USE	Parcels	Hectares	AssessPoints
APT	1	2.59	240
MLT	346	17.04	
SF	1431	77.05	
Vac_Res	24	11.32	
Rea	24	20.29	
Comm	3	9.84	
Inst	5	23.01	
Industrial	0	0.00	
Total	1834	161.14	2073

Hydrant Count: 169

Ductile iron sub-DMA - Orleans

Pipe Count by Diameter

Type	Internal Diameter	Count	Kilometers	% by size
DI	152	12	1.50	50.0%
DI	203	9	0.72	37.5%
DI	305	2	0.07	8.3%
DI	406	1	0.04	4.2%
	Total	24	2.33	100.0%
Undefined	Undefined services	4	0.09	100.0%

Summary Pipe Count by Material Type

Type	Count	Kilometers	% by length
DI	24	2.33	96.3%
PVC	0	0.00	0.0%
Undefined	4	0.09	3.7%
Total	28	2.42	100.0%

Land use by Parcel (with duplicates removed)

LAND_USE	Parcels	Hectares	AssessPoints
APT	0	0.00	0
MLT	154	6.33	
SF	138	7.01	
Vac_Res	0	0.00	
Rea	4	0.00	
Comm	0	0.00	
Inst	2	5.62	
Industrial	0	0.00	
Total	298	18.96	

Hydrant count: 28

Cast iron pipe DMA - Meadowlands

Pipe Count by Diameter

Type	Internal Diameter	Count	Kilometers	% by size	%by Length
CI	152	48	6.5387	55.8%	64.1%
CI	203	28	3.0399	32.6%	29.8%
CI	254	8	0.5684	9.3%	5.6%
CI	305	2	0.0564	2.3%	0.6%
	Total	86	10.2035	100.0%	100.0%
C01	914	1	0.0392	100.0%	100.0%
DI	152	5	0.2965	14.7%	12.4%
DI	203	11	0.7747	32.4%	32.3%
DI	305	18	1.3250	52.9%	55.3%
	Total	34	2.3962	100.0%	100.0%
PVC	203	19	1.17848	63.3%	75.2%
PVC	305	10	0.3676	33.3%	23.4%
PVC	406	1	0.0219	3.3%	1.4%
	Total	30	1.56798	100.0%	100.0%
??	102	2	0.05468	11.1%	4.9%
??	152	12	0.7729	66.7%	69.9%
??	203	3	0.26098	16.7%	23.6%
??	254	1	0.01768	5.6%	1.6%
	Total	18	1.10624	100.0%	100.0%

Summary Pipe Count by Material Type

Type	Count	Kilometers	% by length
??	18	1.1062	7.2%
C01	1	0.0392	0.3%
CI	86	10.2035	66.6%
DI	34	2.3962	15.6%
PVC	30	1.5680	10.2%
Total	169	15.3130	100.0%

Landuse by Parcel (with duplicates removed)

LAND_USE	Count	HA
APT	2	2.90
MLT	414	19.17
SF	459	37.05
Vac_Res	1	0.07
Rea	1	1.55
Comm	25	13.30
Vac_Comm	2	0.34
Rep	1	0.82
Inst	4	4.55
Total	909	79.75

Landuse by Assessment Points

LANDUSE	Count
APT count	447

Hydrant Count: 84

Cast iron pipe sub-DMA (purple area) - Meadowlands

Pipe Count by Diameter

Type	Internal Diameter	Kilometers	%by Length
CI	152	2.5278	81.9%
CI	203	0.5588	18.1%
	Total	3.0866	100.0%
DI	152	0.1524	34.4%
DI	305	0.2911	65.6%
	Total	0.4435	100.0%
PVC	203	0.0826	100.0%
	Total	0.0826	100.0%

Summary Pipe Count by Material Type

CI		3.0866	85.4%
DI		0.4435	12.3%
PVC		0.0826	2.3%
Total		3.6127	100.0%

Landuse

LAND_USE	Count
SF	179
Comm	2
Total	181

Hydrant Count: 21

Cast iron pipe sub-DMA (purple + blue areas) - Meadowlands

Pipe Count by Diameter

Type	Internal Diameter	Kilometers	%by Length
CI	152	2.6722	63.6%
CI	203	1.1265	26.8%
CI	254	0.4015	9.6%
	Total	4.2002	100.0%
DI	152	0.1524	34.4%
DI	305	0.2911	65.6%
	Total	0.4435	100.0%
PVC	203	1.106	100.0%
	Total	1.106	100.0%

Summary Pipe Count by Material Type

CI		4.2002	73.1%
DI		0.4435	7.7%
PVC		1.1060	19.2%
Total		5.7497	100.0%

Landuse

LAND_USE	Count
MLT	110
SF	279
Comm	2
Inst	1
Total	392

Hydrant Count: 34

APPENDIX D

ANALYTICAL MODELING OF MINIMUM NIGHT FLOW COMPONENTS

An analytical model can be developed to separate measured minimum night flows of DMAs into the following 3 components:

a. Background leakage
b. Recoverable leakage
c. Residential night demand

The analytical model is based on the following assumptions:

a. Background leakage is mostly due to leaks at joints and valve fittings and therefore is proportional to the DMA's average night pressure raised to power 1.5 (i.e., $P^{1.5}$).
b. Recoverable leakage is mostly due to fixed-area leaks, i.e., corrosion holes and breaks, and hence is proportional to the DMA's average night pressure raised to power 0.5 (i.e., $P^{0.5}$).
c. Average residential night demand is not dependent on the DMA's pressure (e.g., due to fixed volume toilette flushes).

Let f_i^T be the DMA's total minimum night flow and $f_i^{BL}, f_i^{RL}, f_i^{RD}$ be flow components due to background leakage, recoverable leakage, and residential night demand, respectively, at pressure P_i. If the minimum night flow is measured at three different pressures, P_1, P_2, and P_3, we will have:

$$f_1^T = f_1^{BL} + f_1^{RL} + f_1^{RD}$$

$$f_2^T = f_2^{BL} + f_2^{RL} + f_2^{RD} = f_1^{BL}\left(\frac{P_2}{P_1}\right)^{1.5} + f_1^{RL}\left(\frac{P_2}{P_1}\right)^{0.5} + f_2^{RD}$$

$$f_3^T = f_3^{BL} + f_3^{RL} + f_3^{RD} = f_1^{BL}\left(\frac{P_3}{P_1}\right)^{1.5} + f_1^{RL}\left(\frac{P_3}{P_1}\right)^{0.5} + f_3^{RD}$$

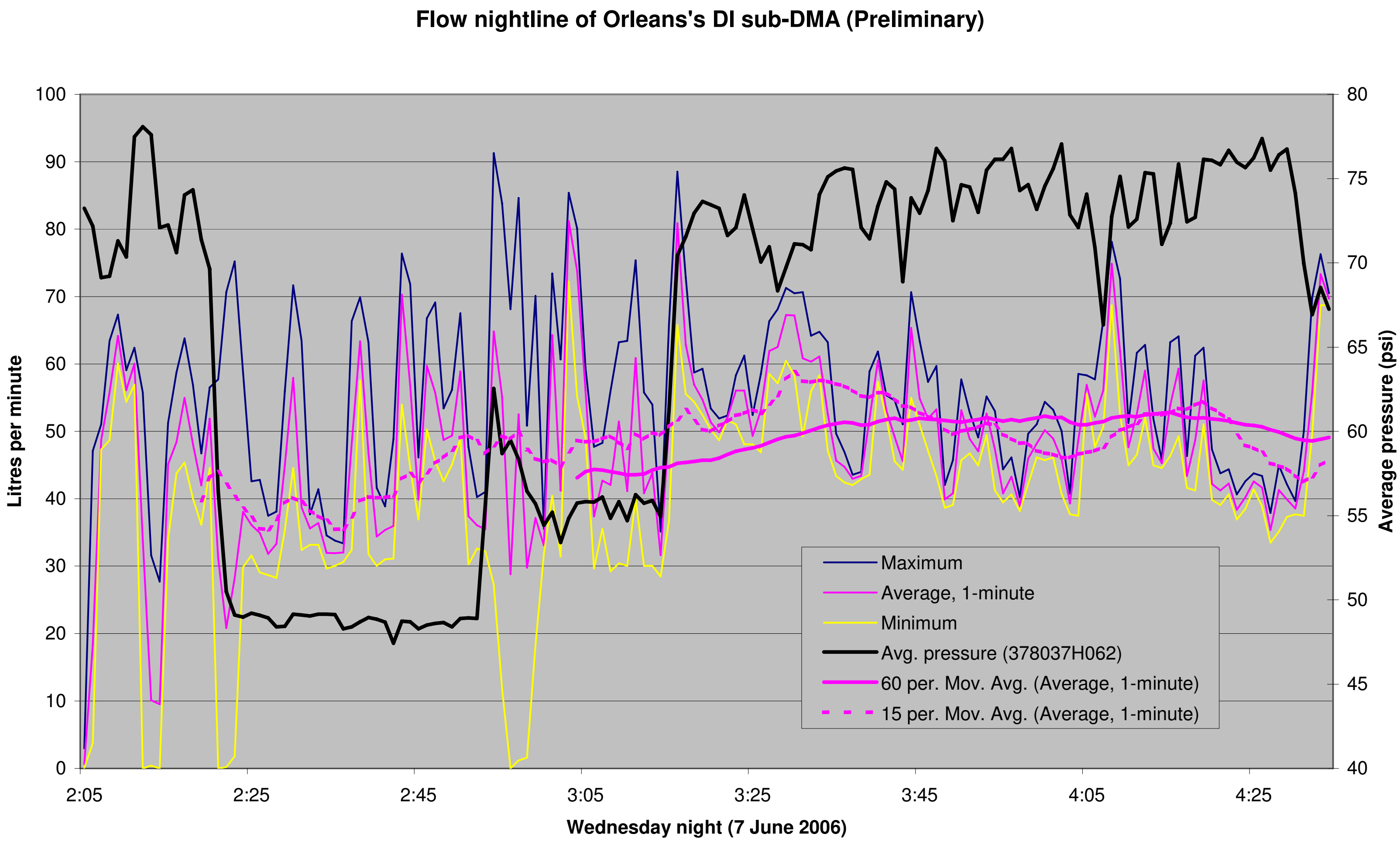
$$f_1^{RD} = f_2^{RD} = f_3^{RD}$$

The above system of three linear equations can be easily solved to determine the following three unknowns: (a) background leakage rate, (b) recoverable leakage rate, and (c) residential night demand.

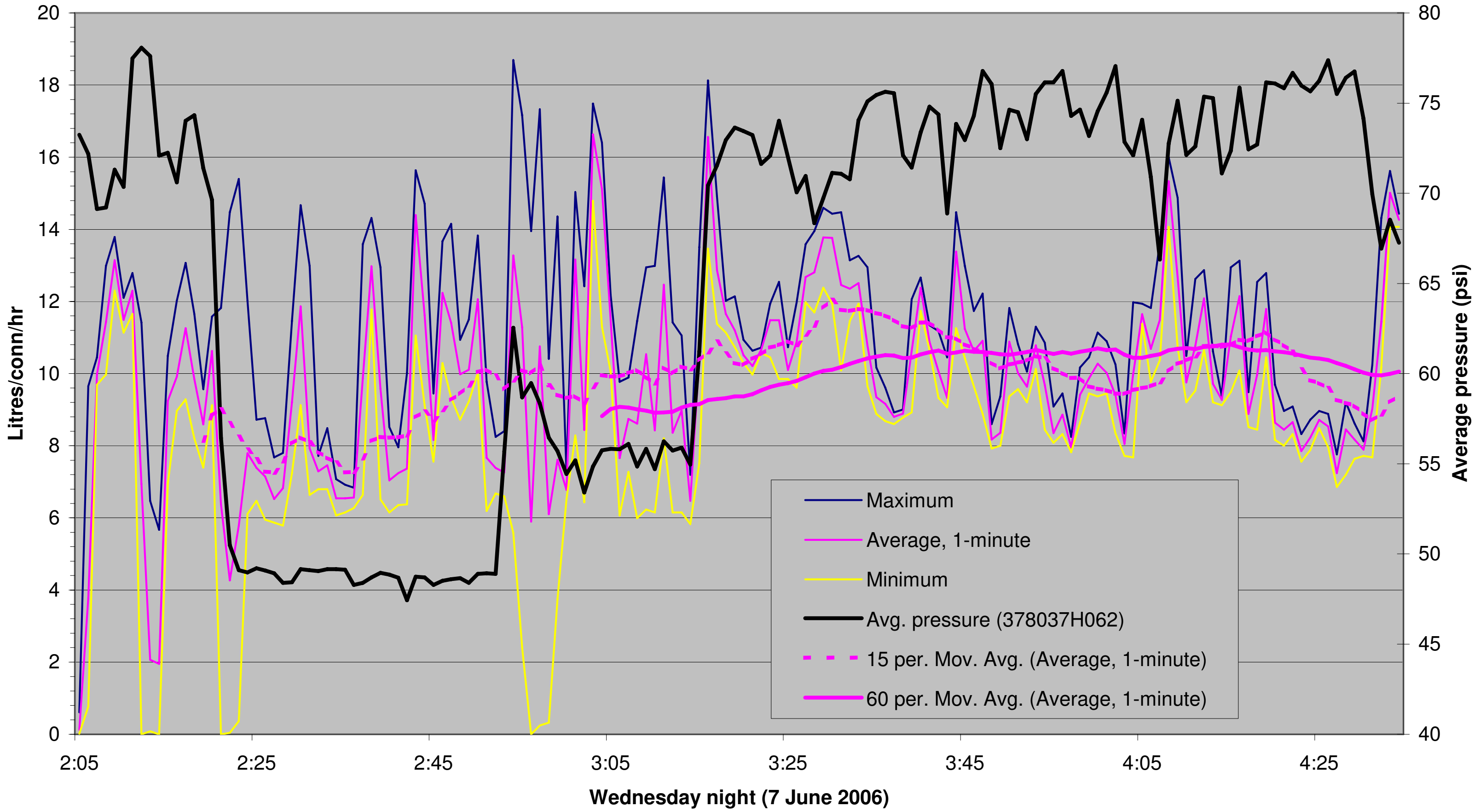
Flow rates in the above equations are assumed to be the minimum 60-minute moving averages between 1 and 5 AM. It should be emphasized that the pressures in the above equations are average values for the whole DMA during flow measurements (i.e., measured at the point corresponding to the DMAs average pressure). Flow meters will have a resolution of 1 litre (i.e., 1 pulse per litre) and the flow will be logged at 1-minute intervals between 1 and 5 AM. Flow measurements will be made over 4 nights, excluding Friday and Saturday nights, with a different pressure for each night (e.g., 20, 40, 60, and 80 psi). Alternatively, though at the risk of lower accuracy, flow measurements may be made over one night only between 1 and 5 AM with a different pressure set over each of the four hours (e.g., 20, 40, 60, and 80 psi). In the latter case, flow rates corresponding to 15-minute rolling averages will be used.

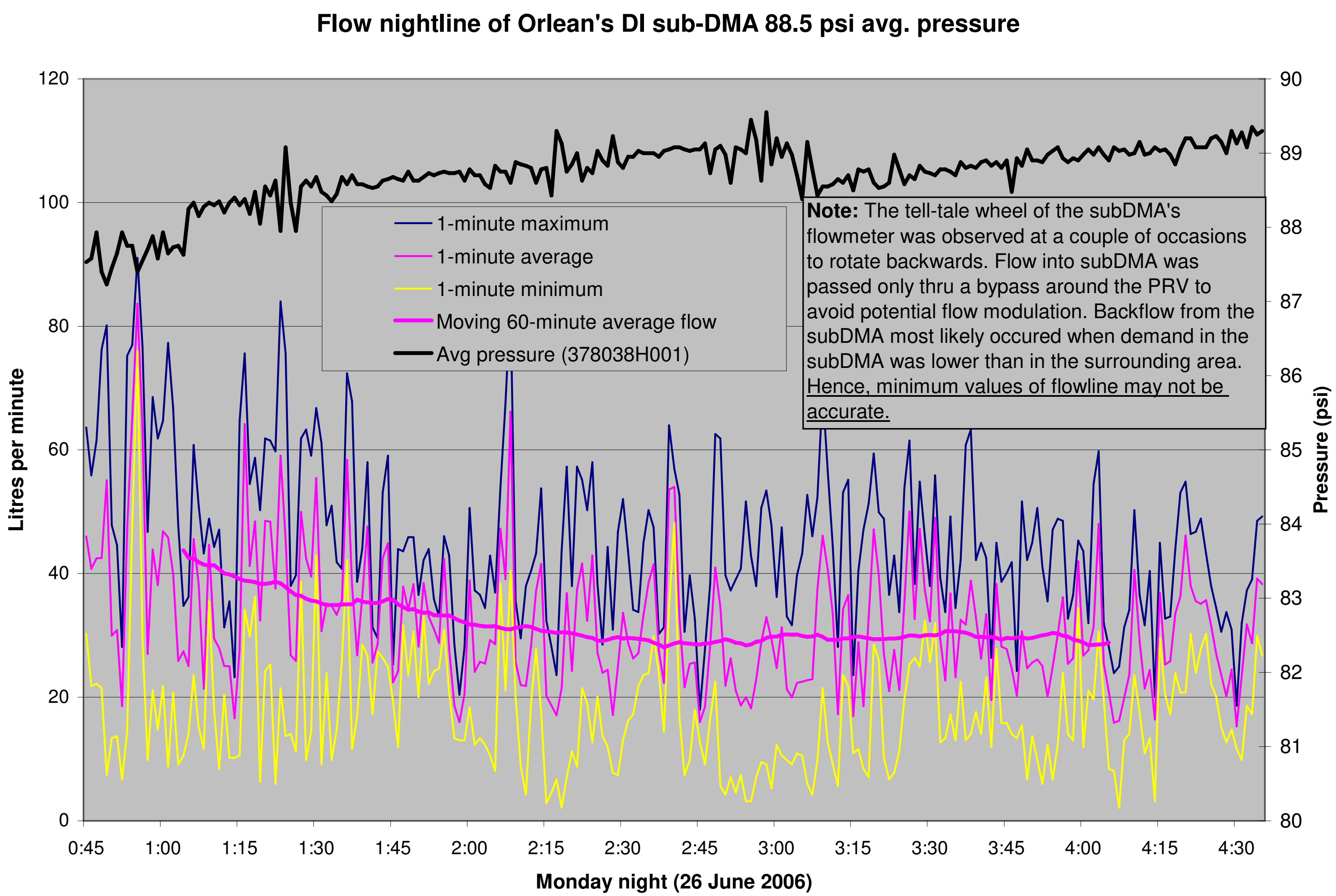
APPENDIX E

FLOW AND PRESSURE NIGHTLINES

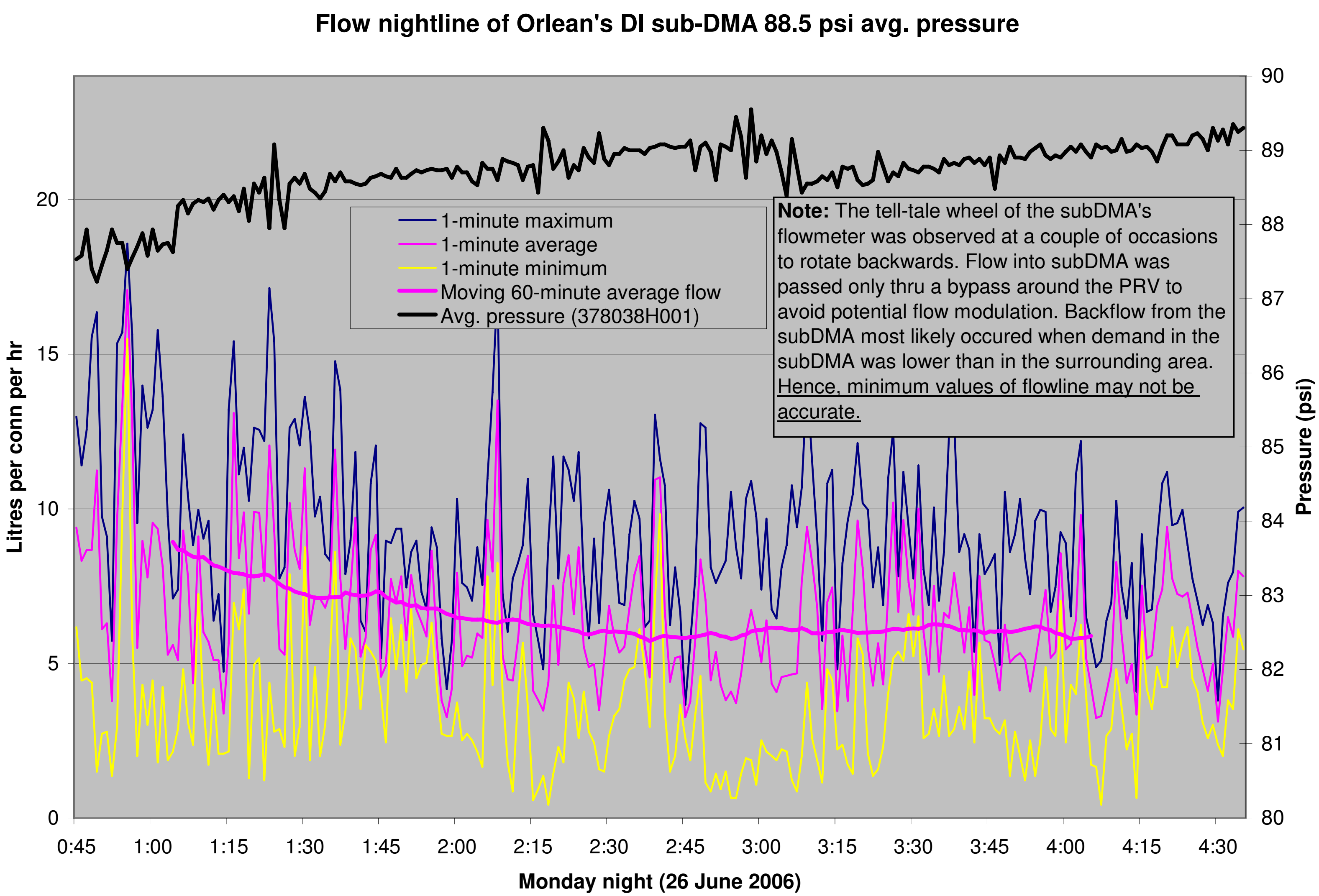

Flow nightline of Orleans's DI sub-DMA (Preliminary)
Litres per minute
Average pressure (psi)
Wednesday night (7 June 2006)
Maximum
Average, 1-minute
Minimum
Avg. pressure (378037H062)
60 per. Mov. Avg. (Average, 1-minute)
15 per. Mov. Avg. (Average, 1-minute)
100
90
80
70
60
50
40
30
20
10
0
80
75
70
65
60
55
50
45
40
2:05
2:25
2:45
3:05
3:25
3:45
4:05
4:25

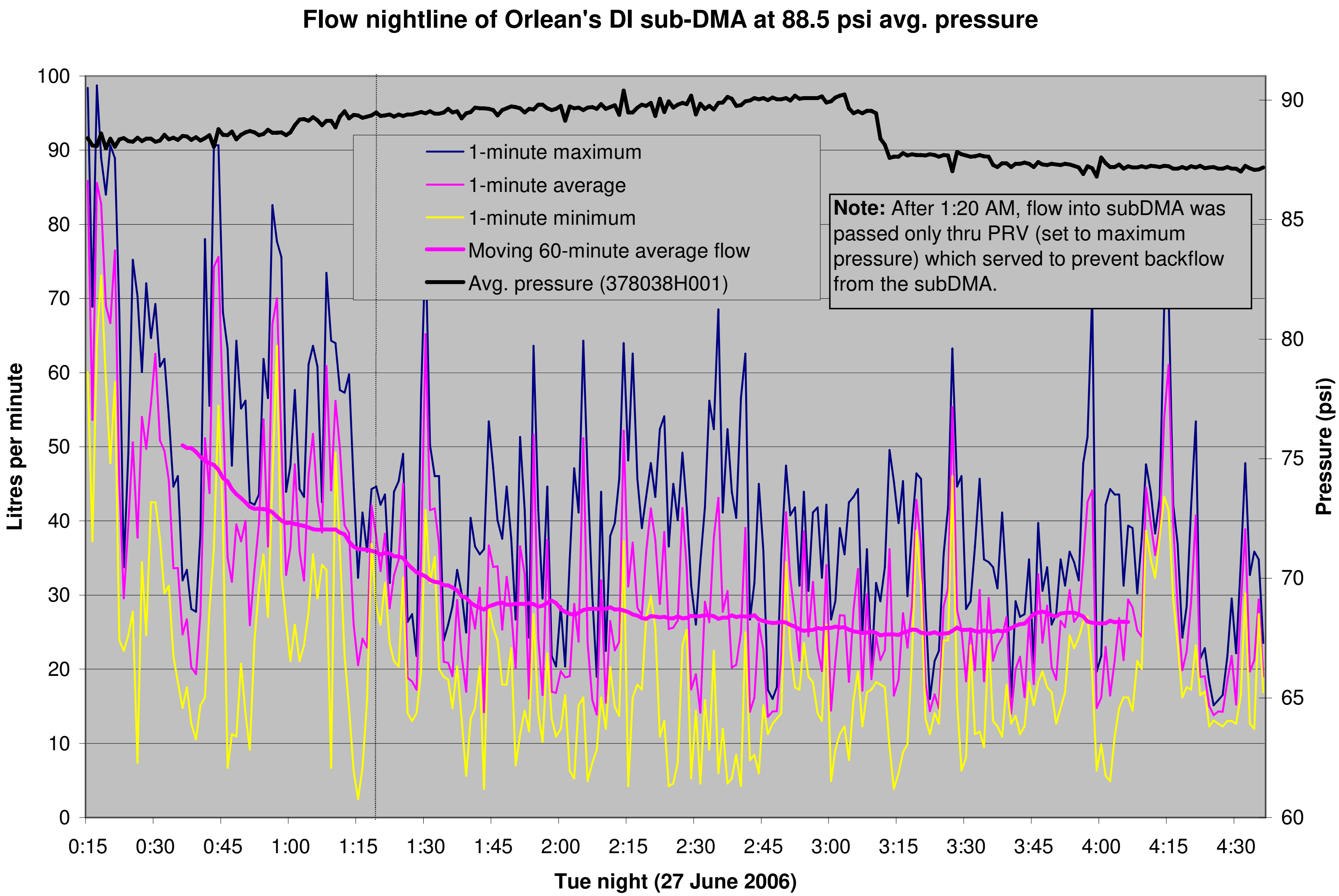
Flow nightline of Orleans's DI sub-DMA (Preliminary)



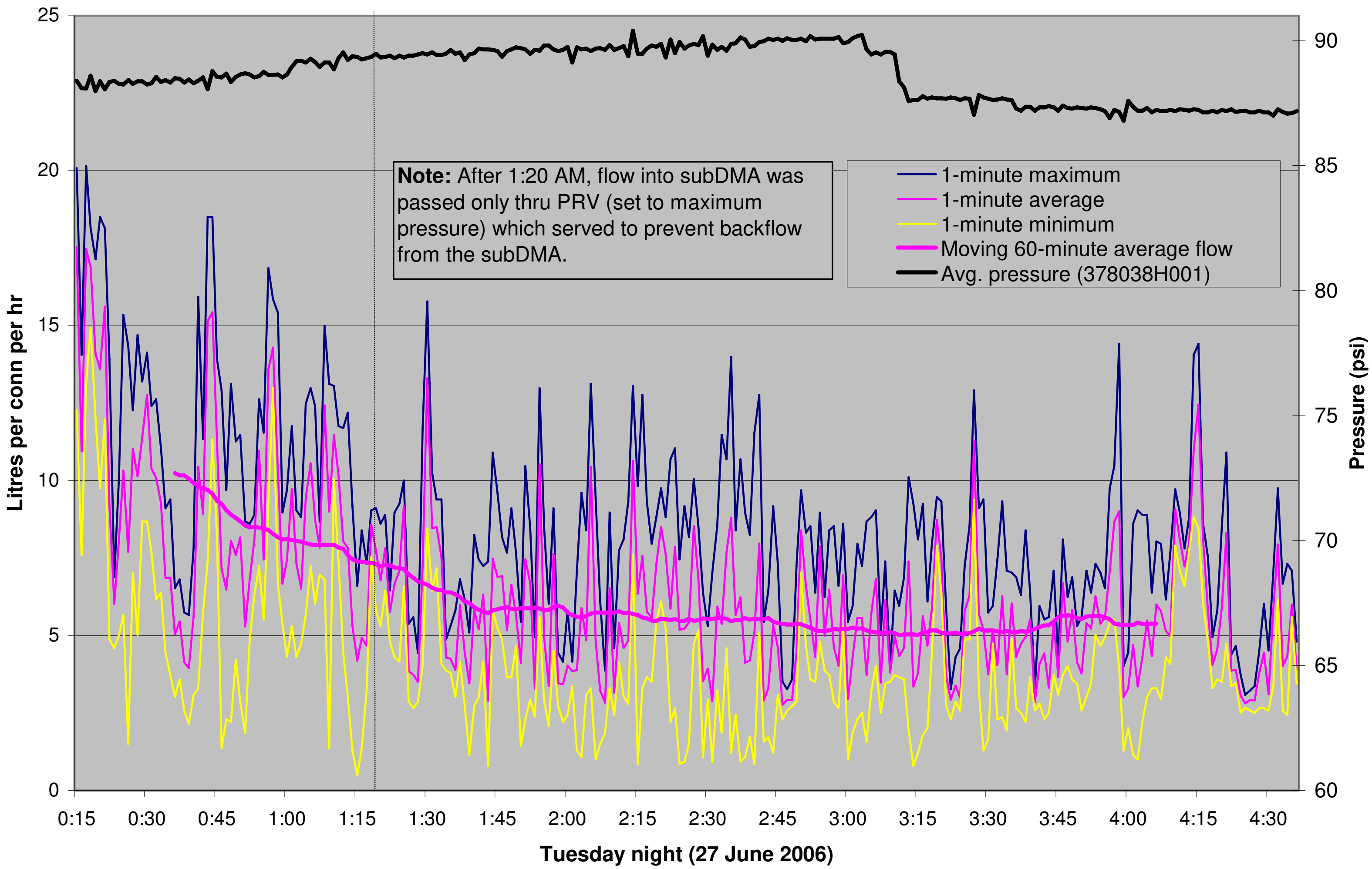


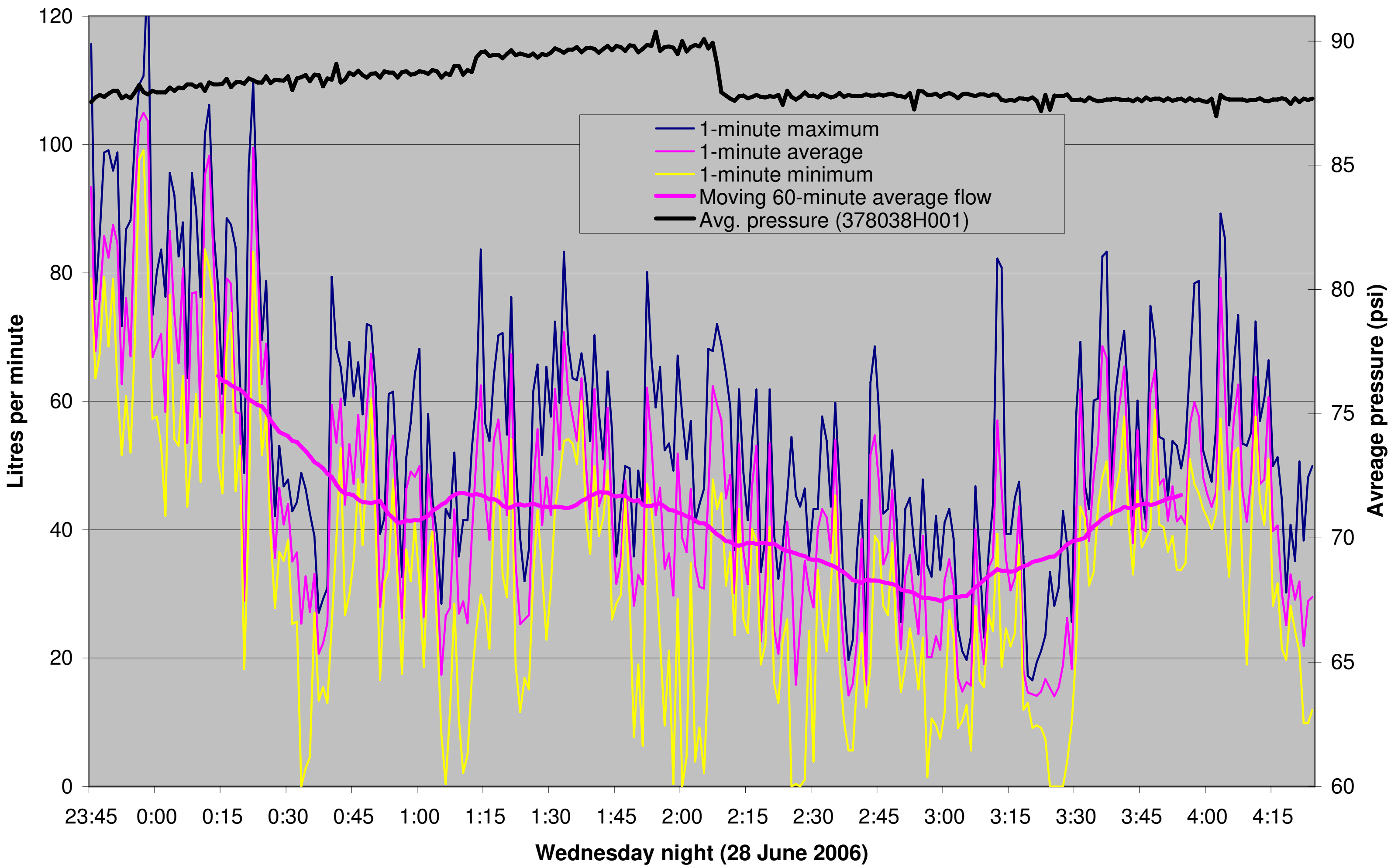
Flow nightline of Orlean's DI sub-DMA 88.5 psi avg. pressure
1-minute maximum
1-minute average
1-minute minimum
Moving 60-minute average flow
Avg pressure (378038H001)
Note: The tell-tale wheel of the subDMA's flowmeter was observed at a couple of occasions to rotate backwards. Flow into subDMA was passed only thru a bypass around the PRV to avoid potential flow modulation. Backflow from the subDMA most likely occured when demand in the subDMA was lower than in the surrounding area. Hence, minimum values of flowline may not be accurate.
Litres per minute
Pressure (psi)
Monday night (26 June 2006)
0
20
40
60
80
100
120
80
81
82
83
84
85
86
87
88
89
90
0:45
1:00
1:15
1:30
1:45
2:00
2:15
2:30
2:45
3:00
3:15
3:30
3:45
4:00
4:15
4:30


Flow nightline of Orlean's DI sub-DMA 88.5 psi avg. pressure
Litres per conn per hr
Pressure (psi)
Monday night (26 June 2006)
1-minute maximum
1-minute average
1-minute minimum
Moving 60-minute average flow
Avg. pressure (378038H001)
Note: The tell-tale wheel of the subDMA's flowmeter was observed at a couple of occasions to rotate backwards. Flow into subDMA was passed only thru a bypass around the PRV to avoid potential flow modulation. Backflow from the subDMA most likely occured when demand in the subDMA was lower than in the surrounding area. Hence, minimum values of flowline may not be accurate.
0
5
10
15
20
80
81
82
83
84
85
86
87
88
89
90
0:45
1:00
1:15
1:30
1:45
2:00
2:15
2:30
2:45
3:00
3:15
3:30
3:45
4:00
4:15
4:30

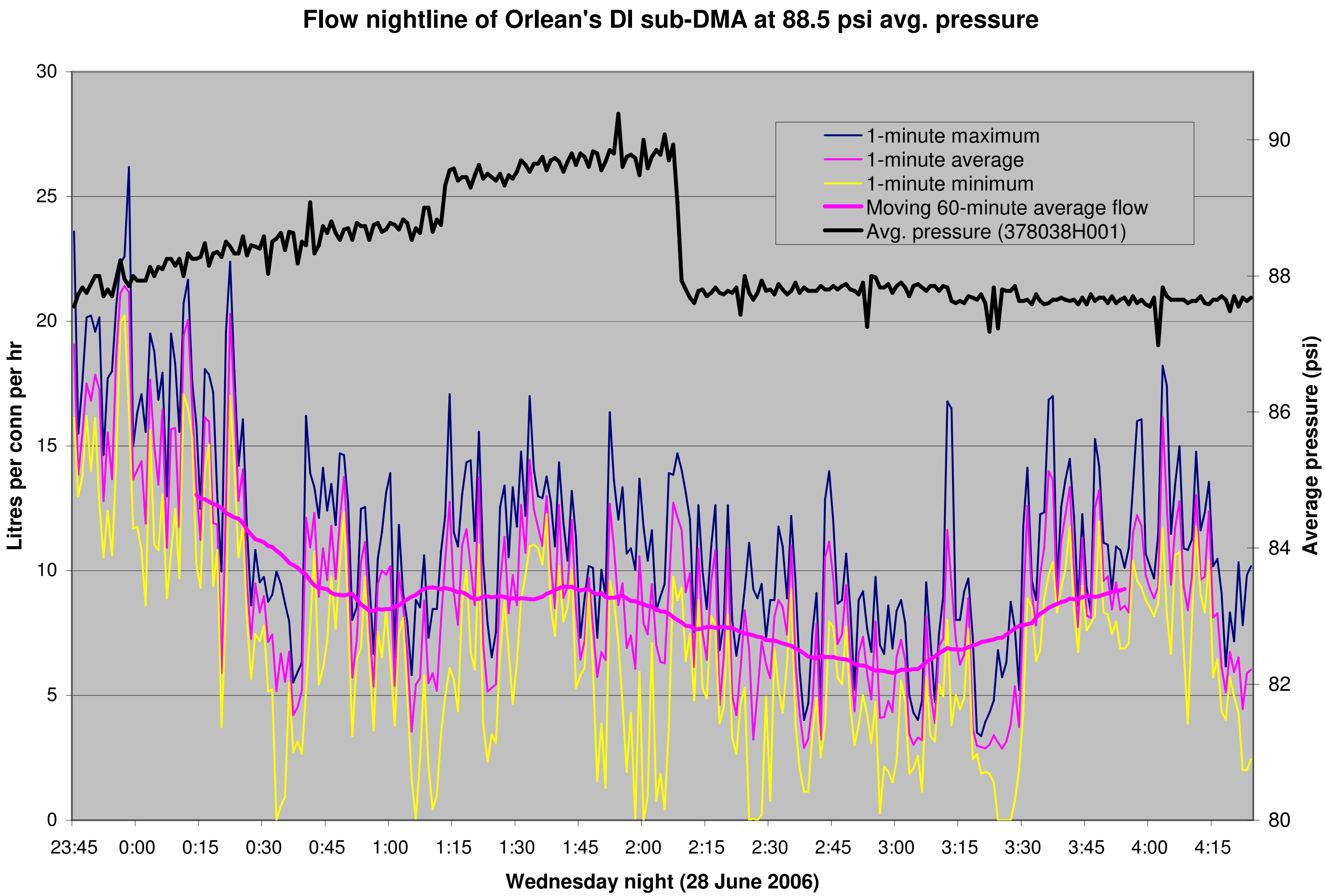

Flow nightline of Orlean's DI sub-DMA at 88.5 psi avg. pressure
Litres per minute
Pressure (psi)
Tue night (27 June 2006)
1-minute maximum
1-minute average
1-minute minimum
Moving 60-minute average flow
Avg. pressure (378038H001)
Note: After 1:20 AM, flow into subDMA was passed only thru PRV (set to maximum pressure) which served to prevent backflow from the subDMA.
0
10
20
30
40
50
60
70
80
90
100
60
65
70
75
80
85
90
0:15
0:30
0:45
1:00
1:15
1:30
1:45
2:00
2:15
2:30
2:45
3:00
3:15
3:30
3:45
4:00
4:15
4:30

Flow nightline of Orlean's DI sub-DMA at 88.5 psi avg. pressure



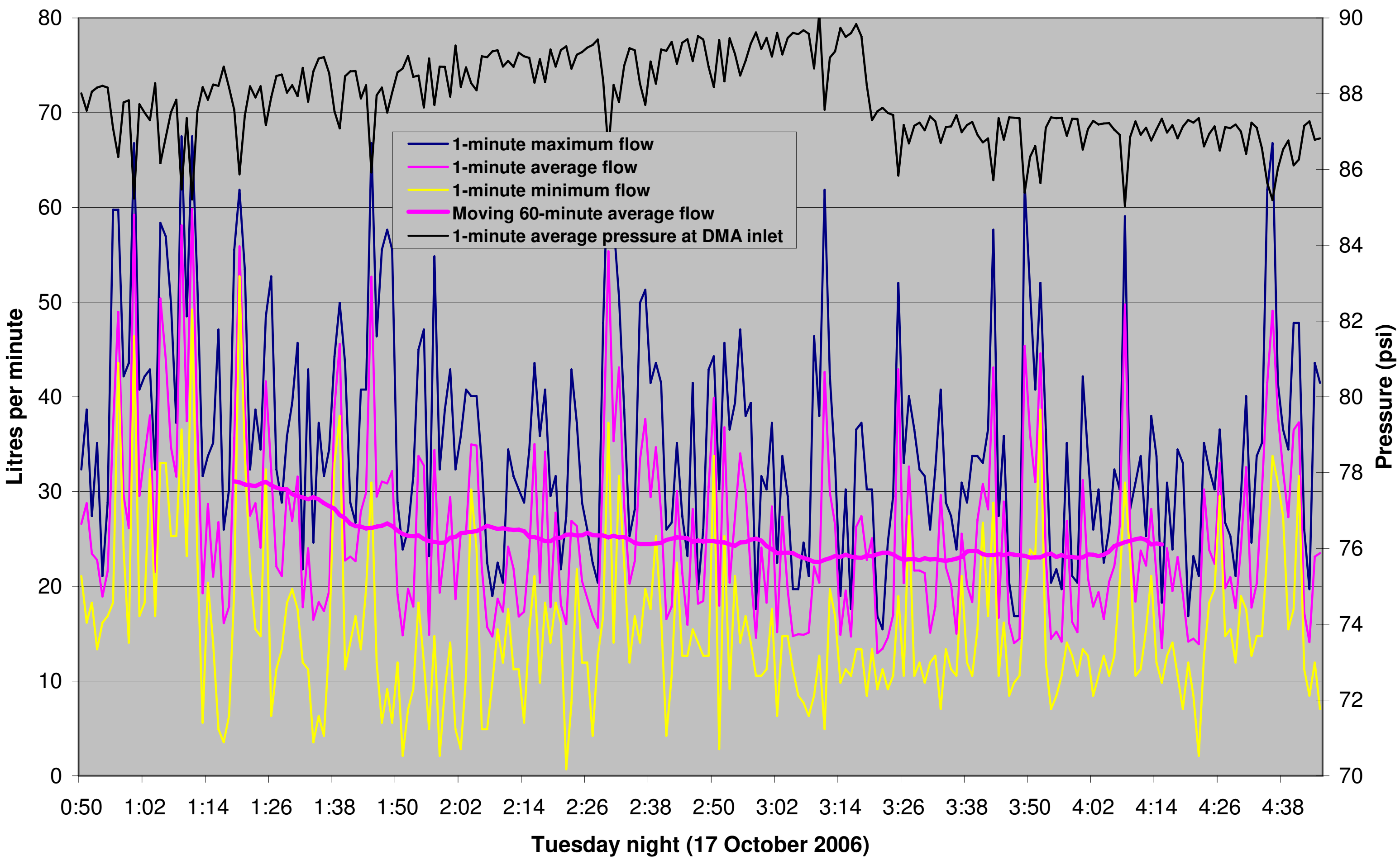


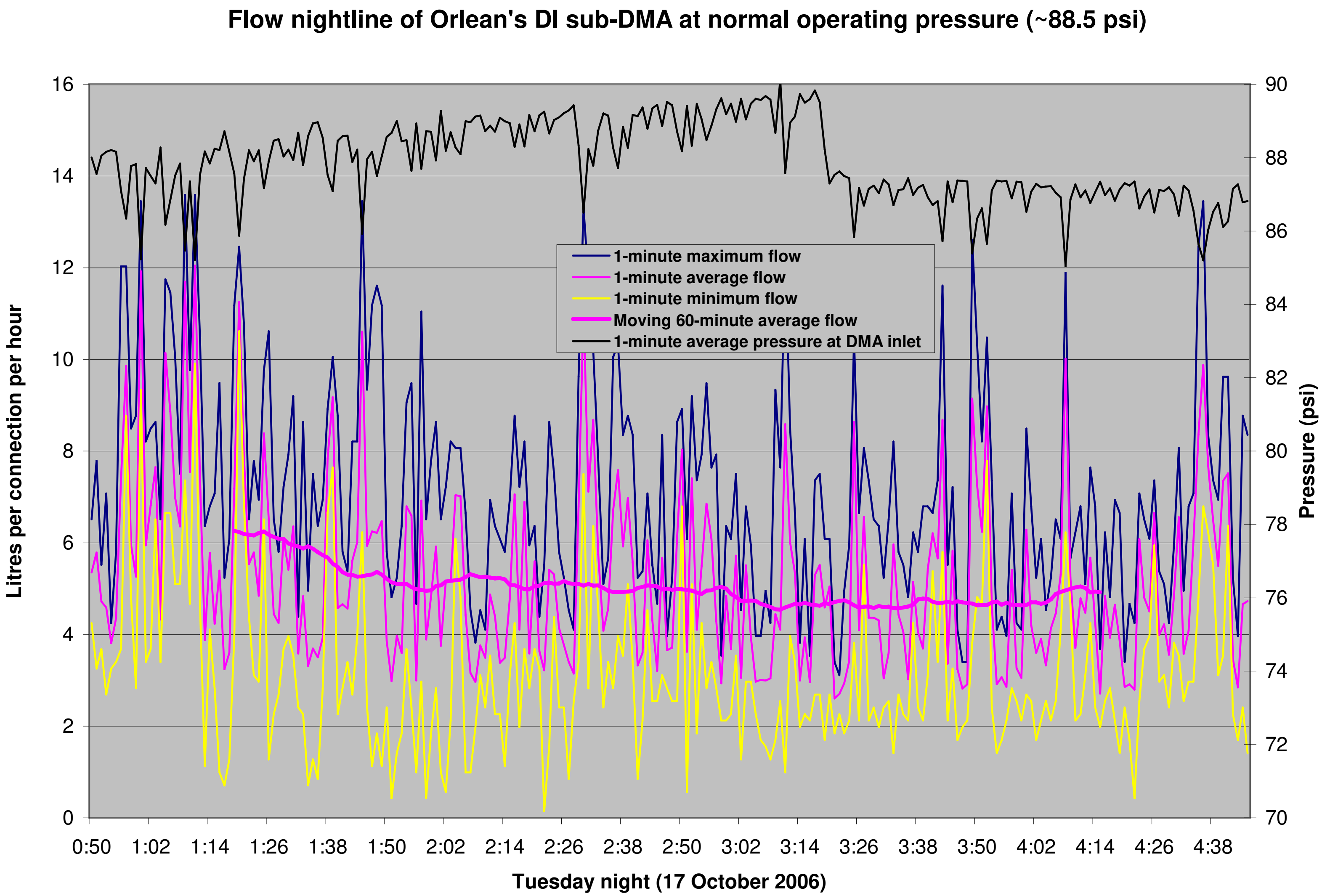
Flow nightline of Orlean's DI sub-DMA at 88.5 psi avg. pressure



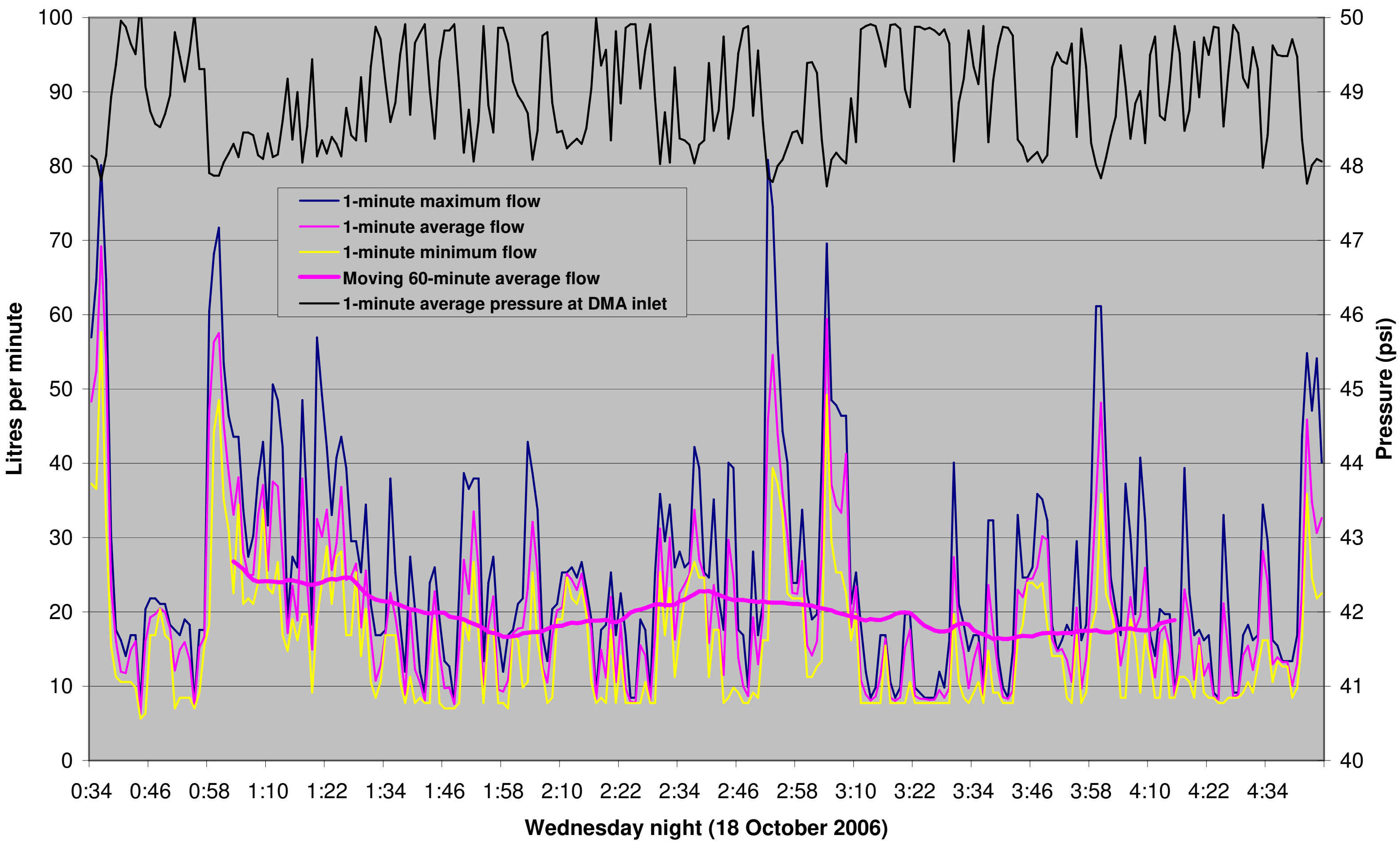
Flow nightline of Orlean's DI sub-DMA at 88.5 psi avg. pressure
1-minute maximum
1-minute average
1-minute minimum
Moving 60-minute average flow
Avg. pressure (378038H001)
Litres per conn per hr
Average pressure (psi)
Wednesday night (28 June 2006)
0
5
10
15
20
25
30
80
82
84
86
88
90
23:45
0:00
0:15
0:30
0:45
1:00
1:15
1:30
1:45
2:00
2:15
2:30
2:45
3:00
3:15
3:30
3:45
4:00
4:15

Flow nightline of Orlean's DI sub-DMA at normal operating pressure (~88.5 psi)

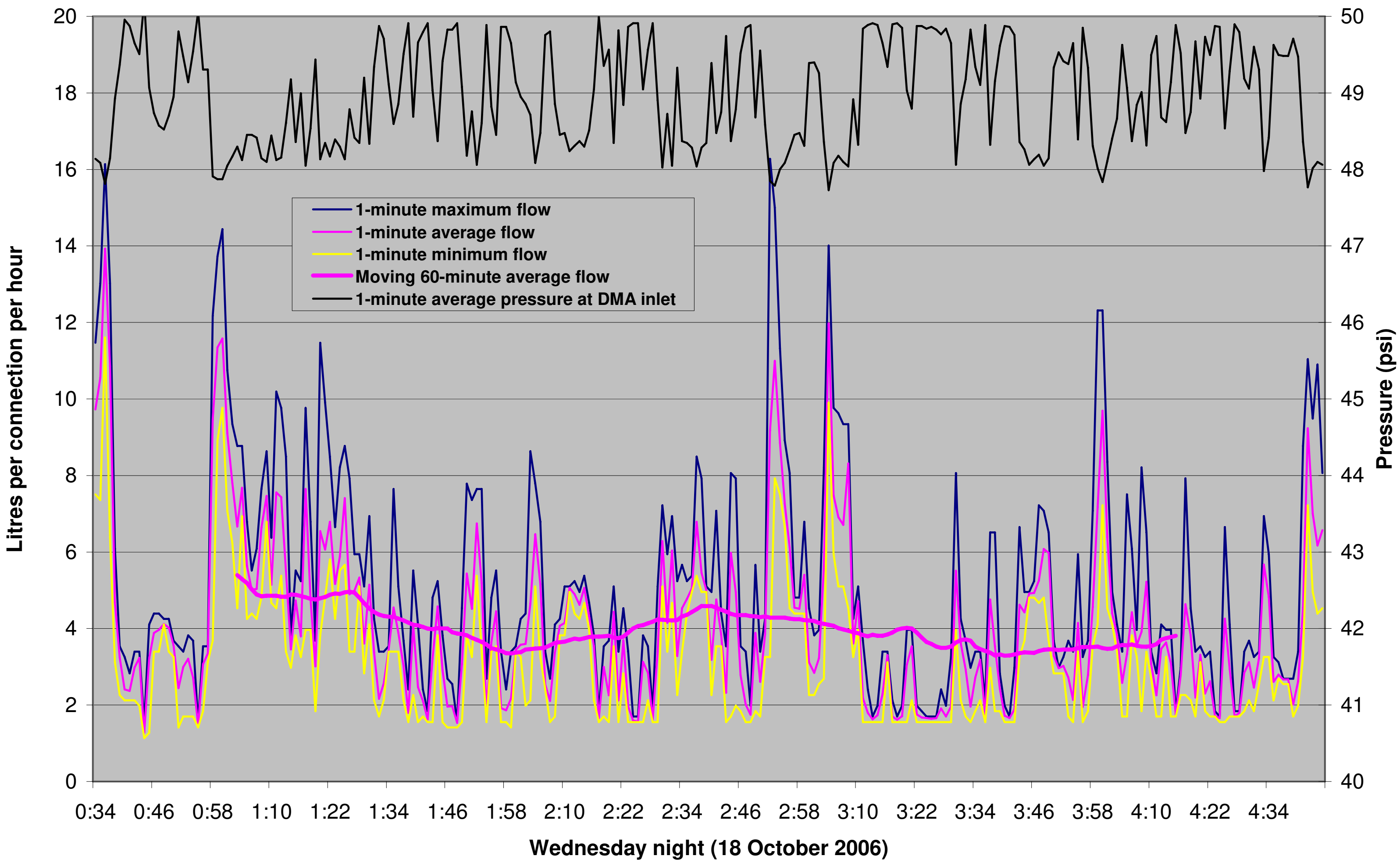



Flow nightline of Orlean's DI sub-DMA at normal operating pressure (~88.5 psi)
Litres per connection per hour
Pressure (psi)
Tuesday night (17 October 2006)
1-minute maximum flow
1-minute average flow
1-minute minimum flow
Moving 60-minute average flow
1-minute average pressure at DMA inlet
0
2
4
6
8
10
12
14
16
70
72
74
76
78
80
82
84
86
88
90
0:50
1:02
1:14
1:26
1:38
1:50
2:02
2:14
2:26
2:38
2:50
3:02
3:14
3:26
3:38
3:50
4:02
4:14
4:26
4:38

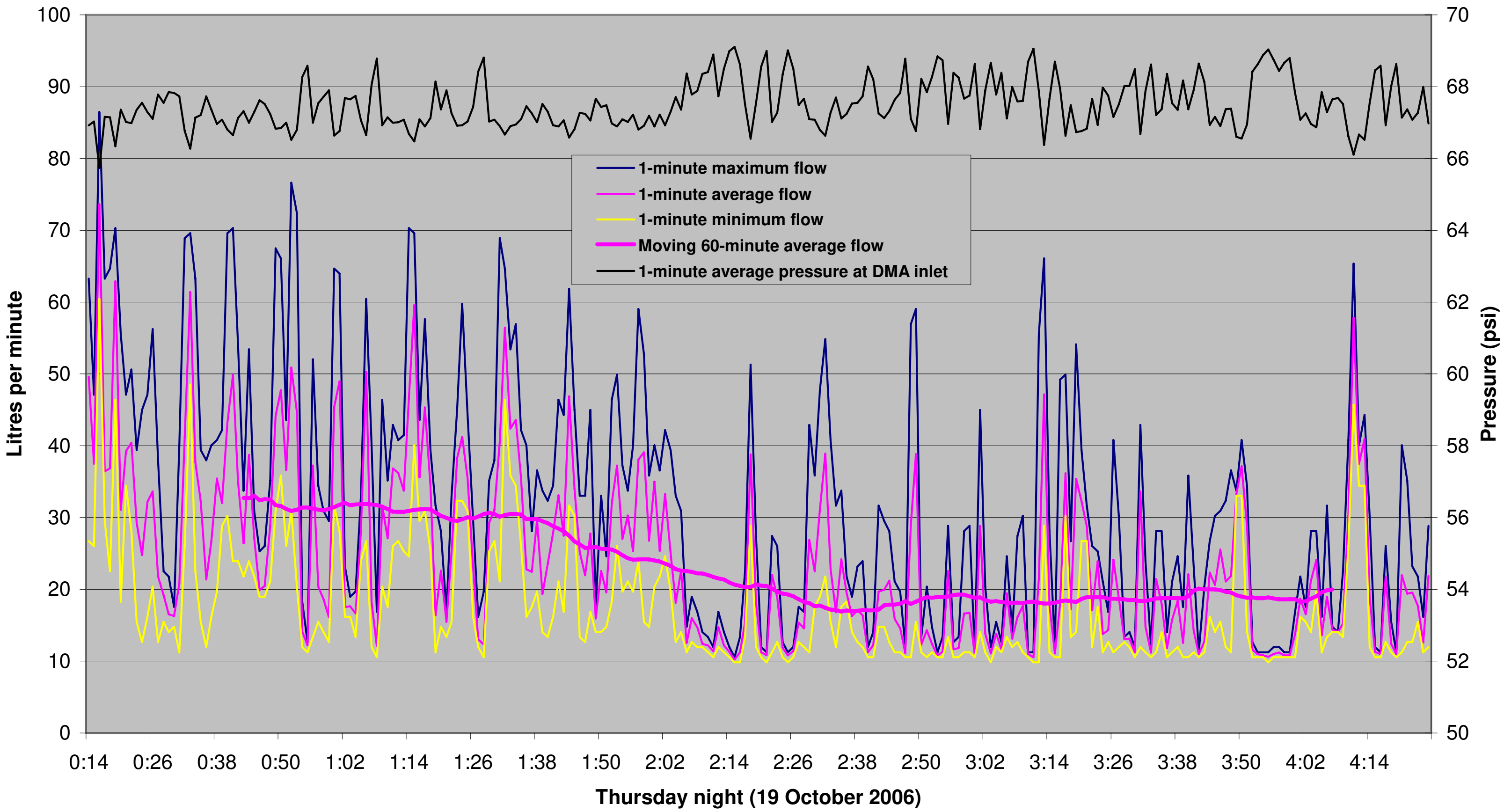
Flow nightline of Orlean's DI sub-DMA at reduced pressure of ~49 psi

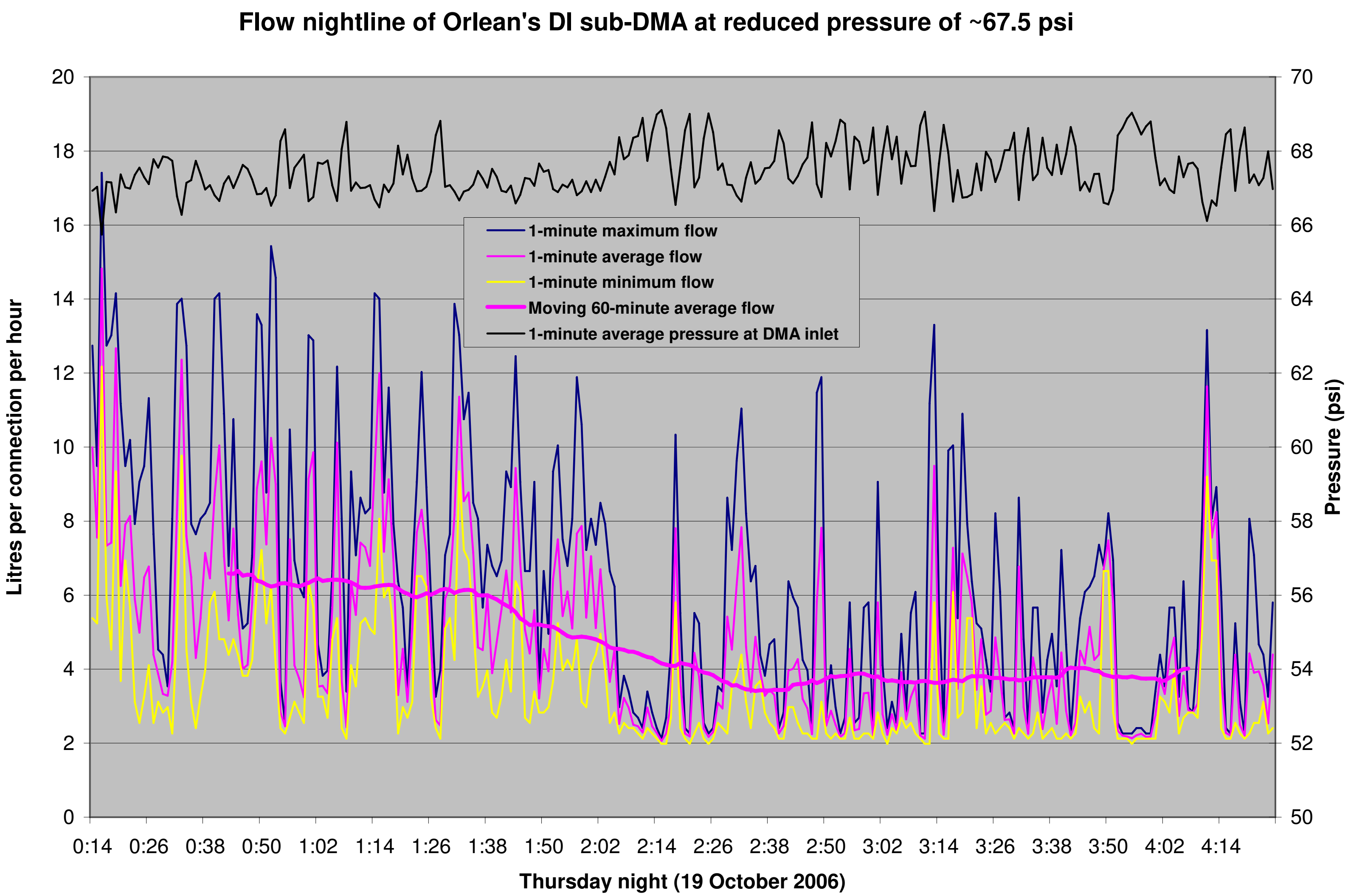


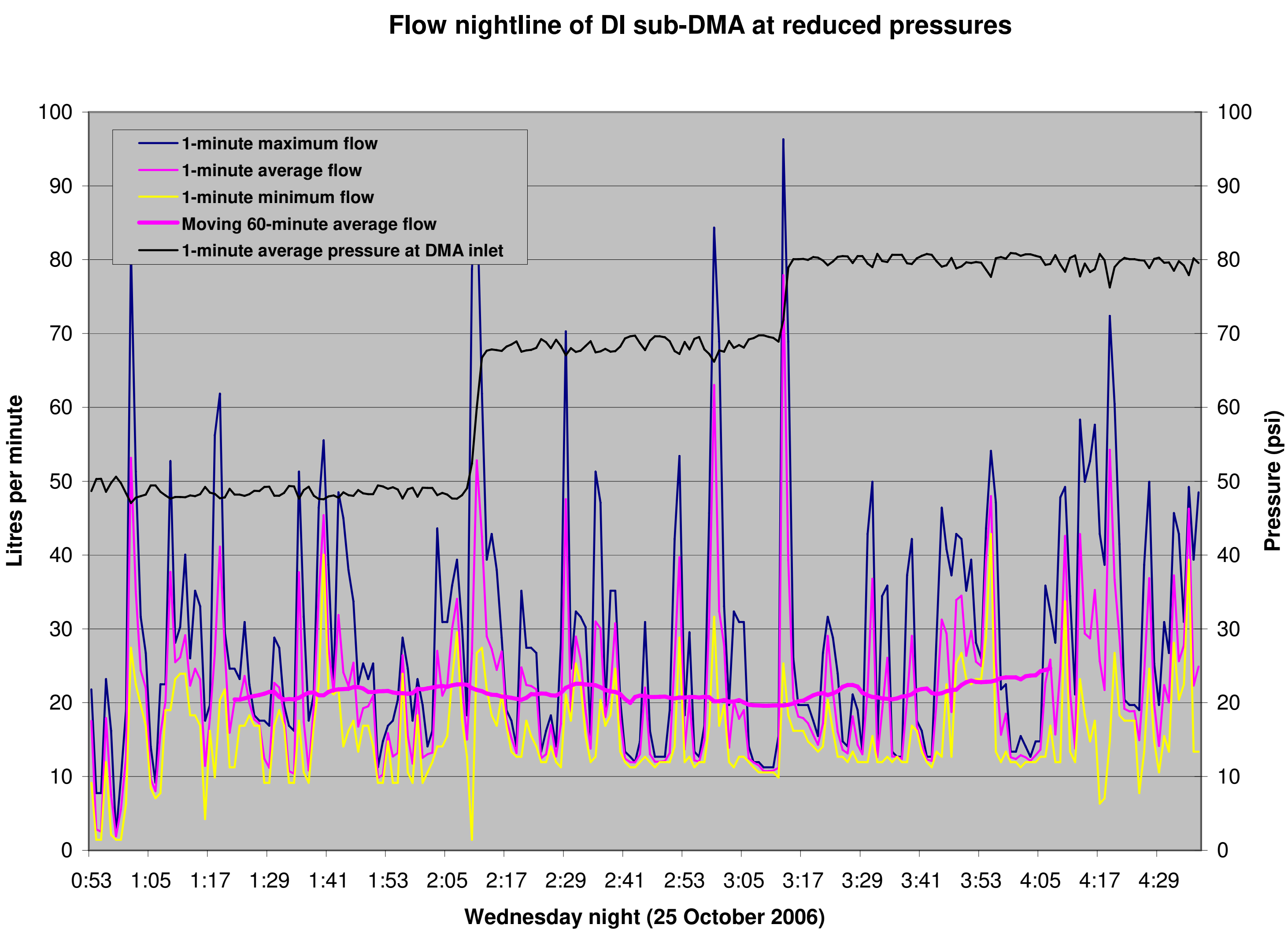
Flow nightline of Orlean's DI sub-DMA at reduced pressure of ~49 psi



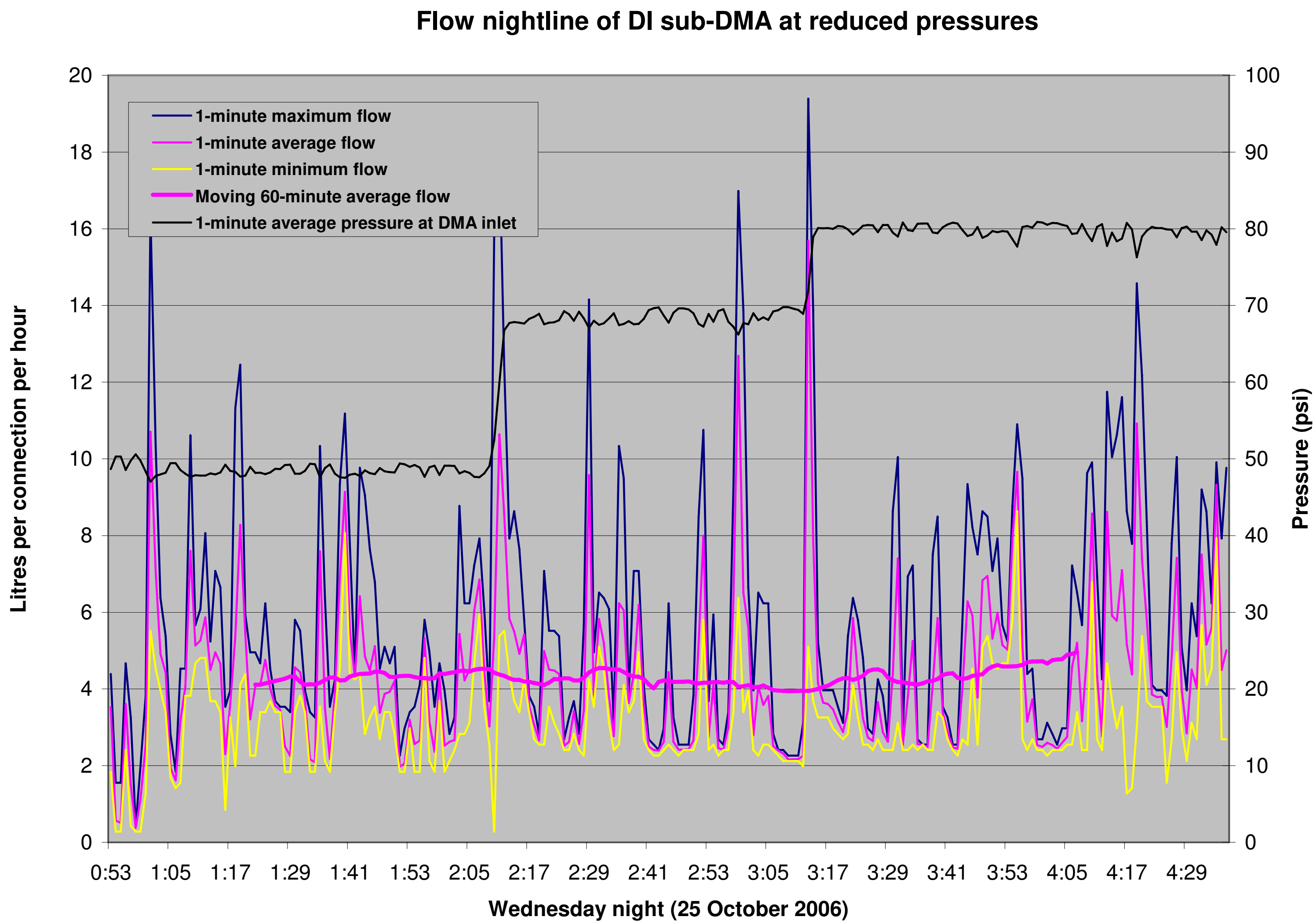
Flow nightline of Orlean's DI sub-DMA at reduced pressure of ~67.5 psi

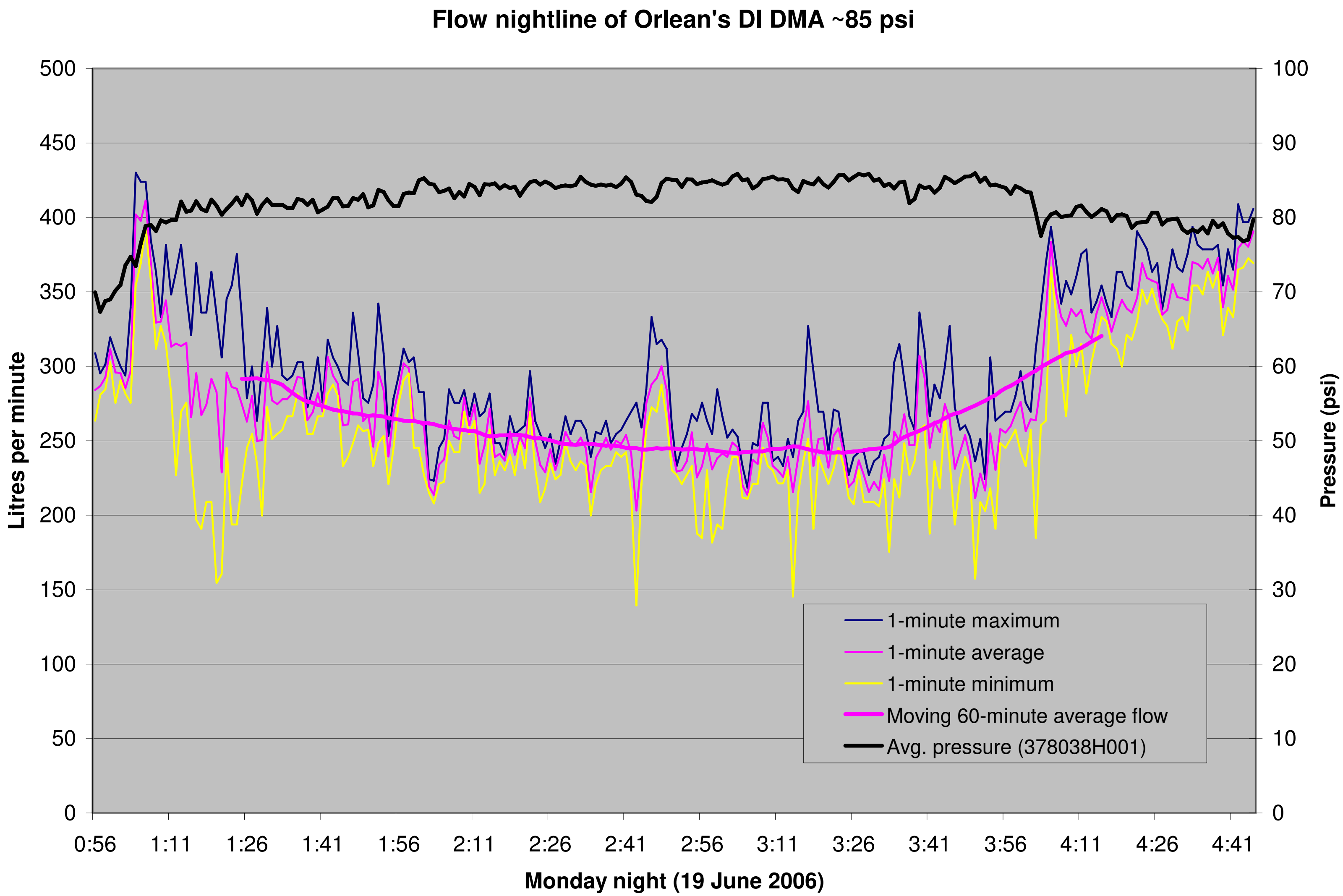



Flow nightline of Orlean's DI sub-DMA at reduced pressure of ~67.5 psi
1-minute maximum flow
1-minute average flow
1-minute minimum flow
Moving 60-minute average flow
1-minute average pressure at DMA inlet
Litres per connection per hour
Pressure (psi)
Thursday night (19 October 2006)
0
2
4
6
8
10
12
14
16
18
20
50
52
54
56
58
60
62
64
66
68
70
0:14
0:26
0:38
0:50
1:02
1:14
1:26
1:38
1:50
2:02
2:14
2:26
2:38
2:50
3:02
3:14
3:26
3:38
3:50
4:02
4:14

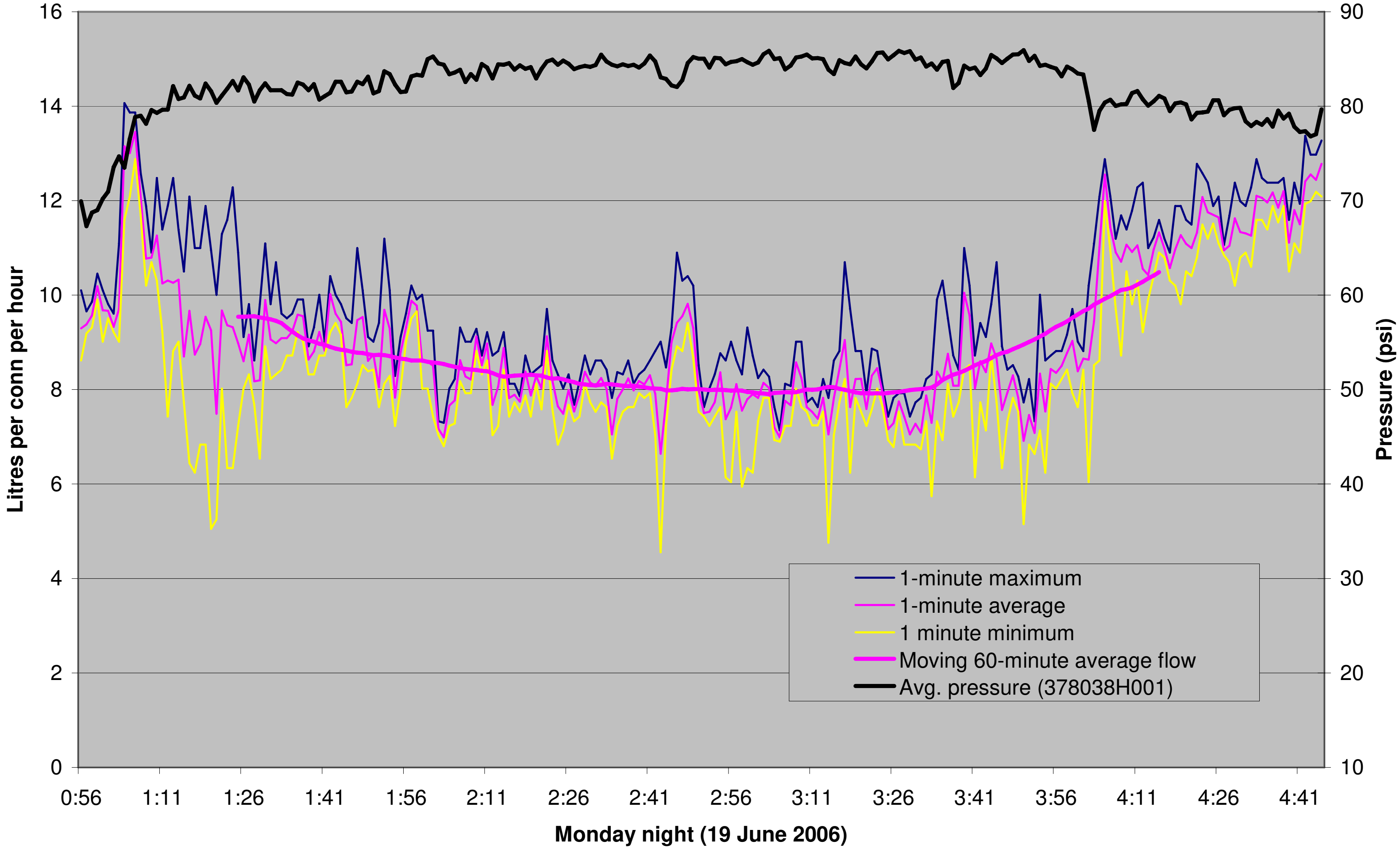


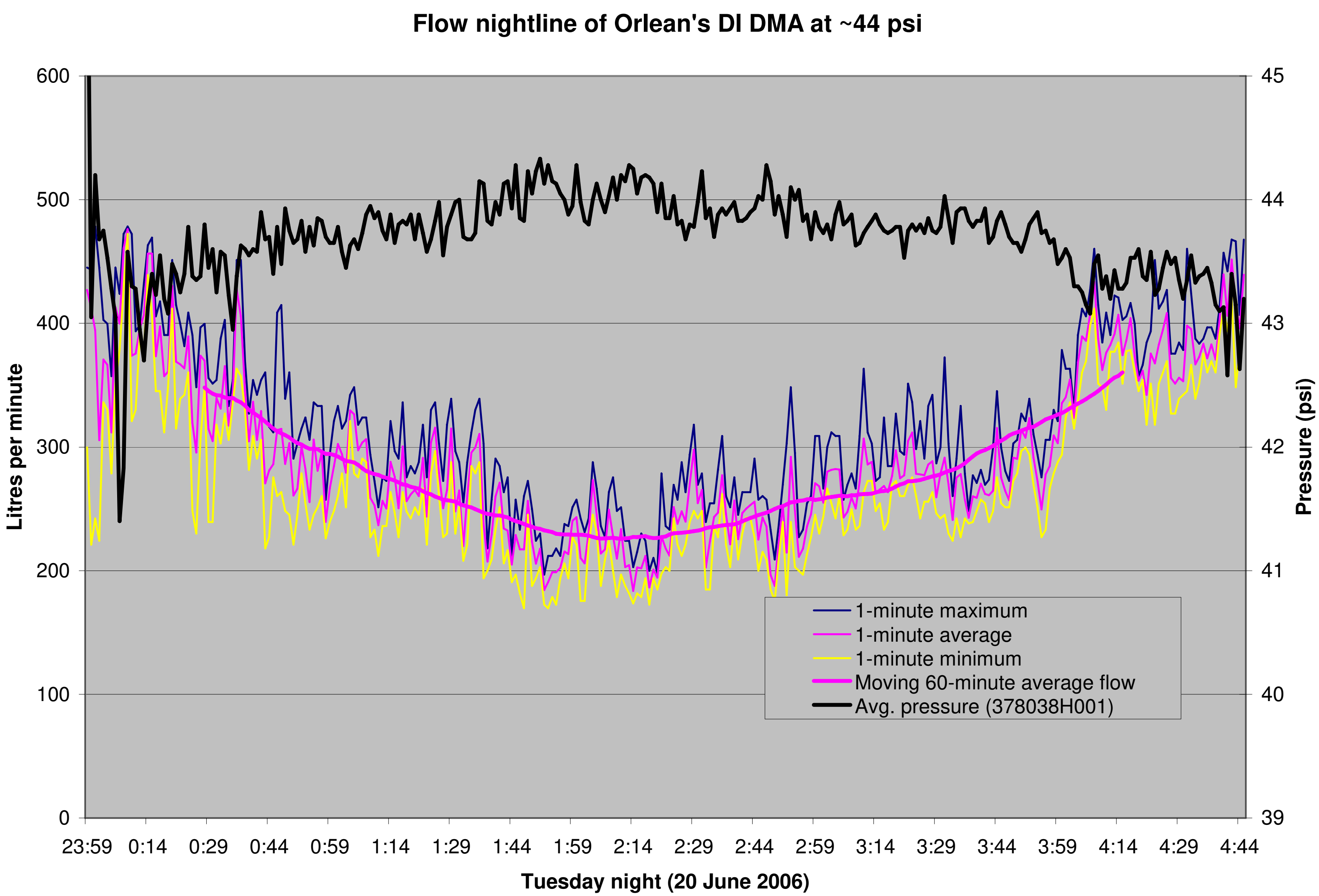
Flow nightline of DI sub-DMA at reduced pressures
1-minute maximum flow
1-minute average flow
1-minute minimum flow
Moving 60-minute average flow
1-minute average pressure at DMA inlet
Litres per minute
Pressure (psi)
Wednesday night (25 October 2006)
0:53
1:05
1:17
1:29
1:41
1:53
2:05
2:17
2:29
2:41
2:53
3:05
3:17
3:29
3:41
3:53
4:05
4:17
4:29


Flow nightline of DI sub-DMA at reduced pressures
1-minute maximum flow
1-minute average flow
1-minute minimum flow
Moving 60-minute average flow
1-minute average pressure at DMA inlet
Litres per connection per hour
Pressure (psi)
Wednesday night (25 October 2006)
0 2 4 6 8 10 12 14 16 18 20
0 10 20 30 40 50 60 70 80 90 100
0:53 1:05 1:17 1:29 1:41 1:53 2:05 2:17 2:29 2:41 2:53 3:05 3:17 3:29 3:41 3:53 4:05 4:17 4:29

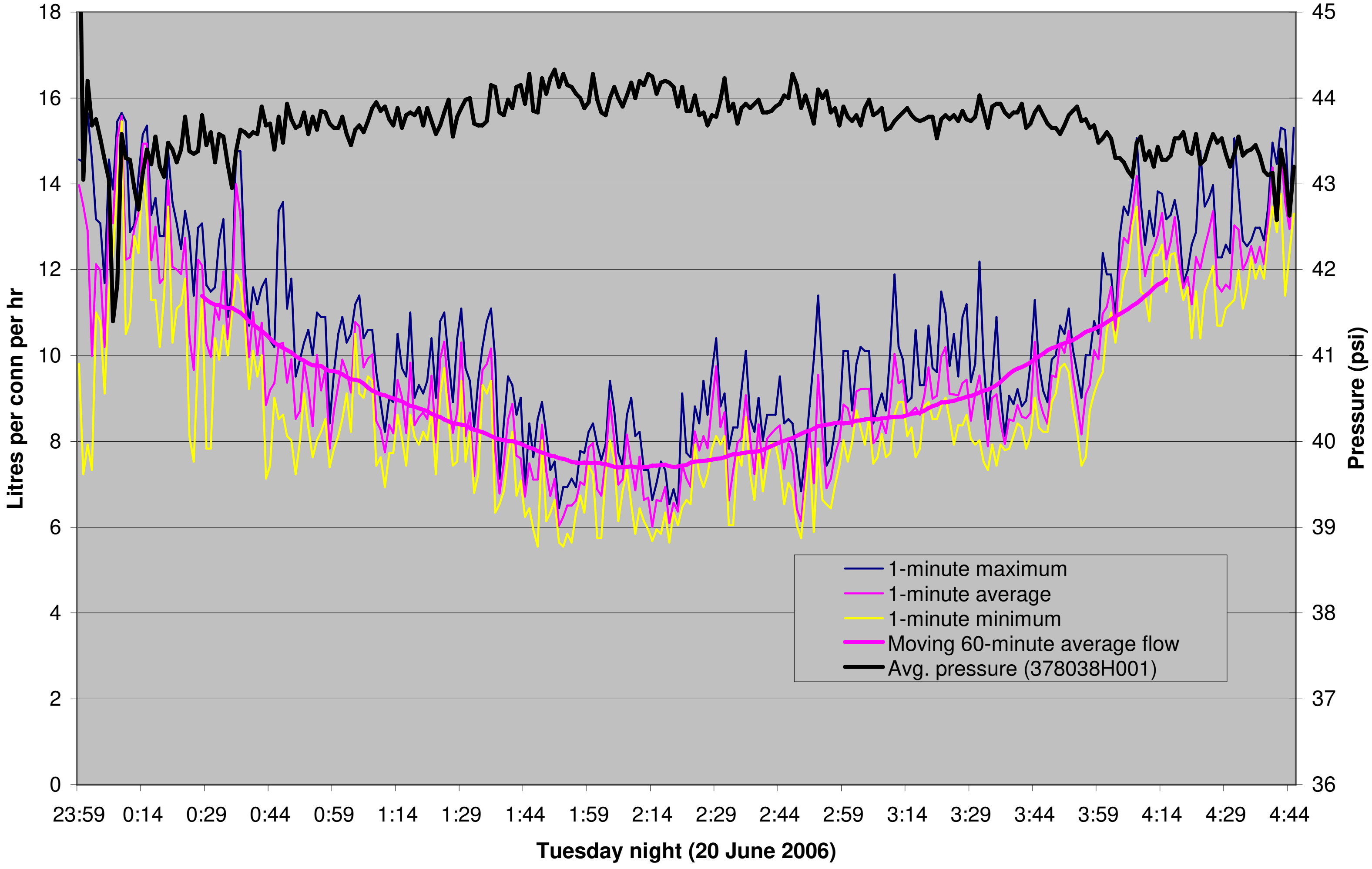

Flow nightline of Orlean's DI DMA ~85 psi
Litres per minute
Pressure (psi)
Monday night (19 June 2006)
0:56
1:11
1:26
1:41
1:56
2:11
2:26
2:41
2:56
3:11
3:26
3:41
3:56
4:11
4:26
4:41
0
50
100
150
200
250
300
350
400
450
500
0
10
20
30
40
50
60
70
80
90
100
1-minute maximum
1-minute average
1-minute minimum
Moving 60-minute average flow
Avg. pressure (378038H001)

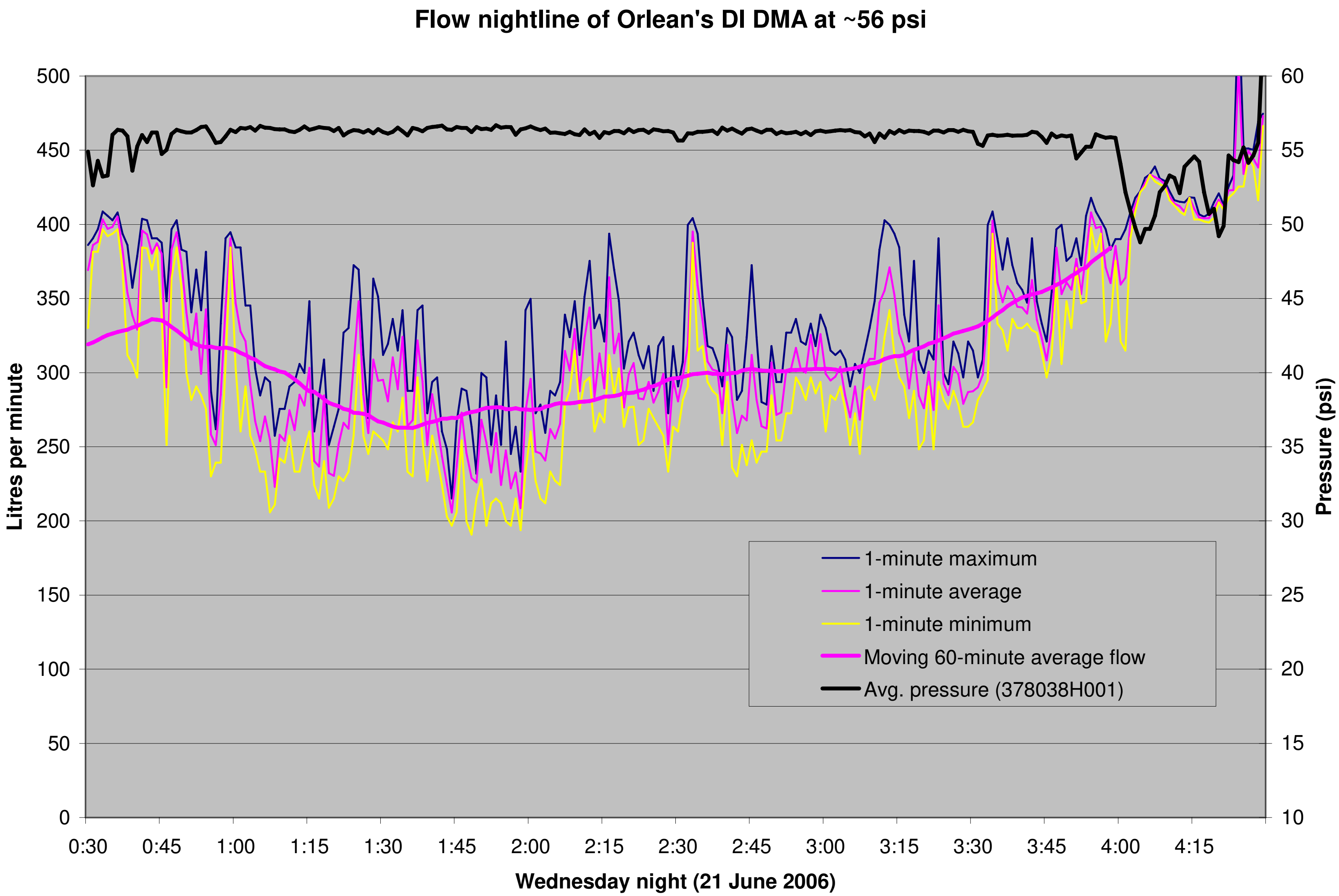
Flow nightline of Orlean's DI DMA at ~85 psi



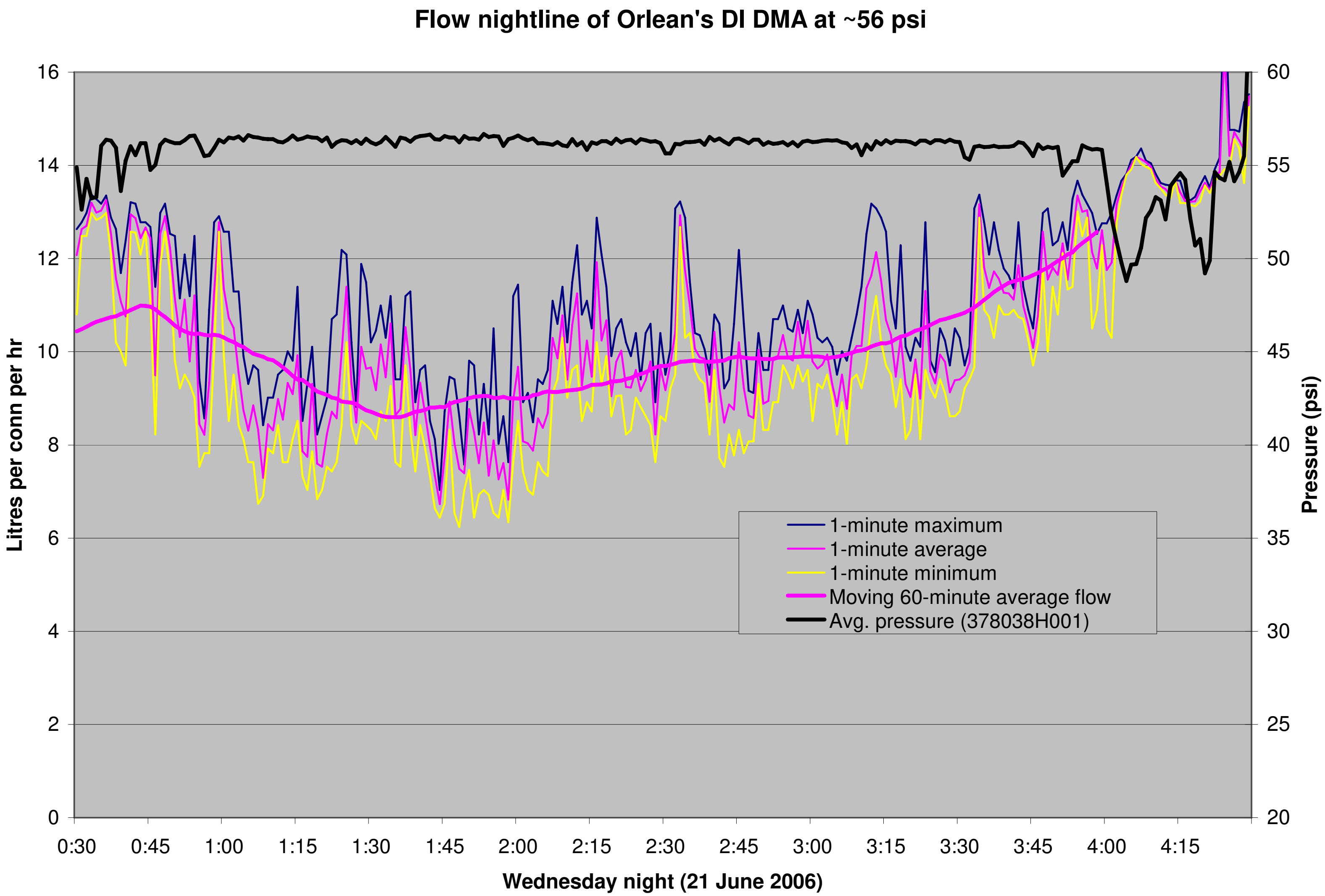

Flow nightline of Orlean's DI DMA at ~44 psi
Litres per minute
Pressure (psi)
600
500
400
300
200
100
0
45
44
43
42
41
40
39
23:59
0:14
0:29
0:44
0:59
1:14
1:29
1:44
1:59
2:14
2:29
2:44
2:59
3:14
3:29
3:44
3:59
4:14
4:29
4:44
Tuesday night (20 June 2006)
1-minute maximum
1-minute average
1-minute minimum
Moving 60-minute average flow
Avg. pressure (378038H001)

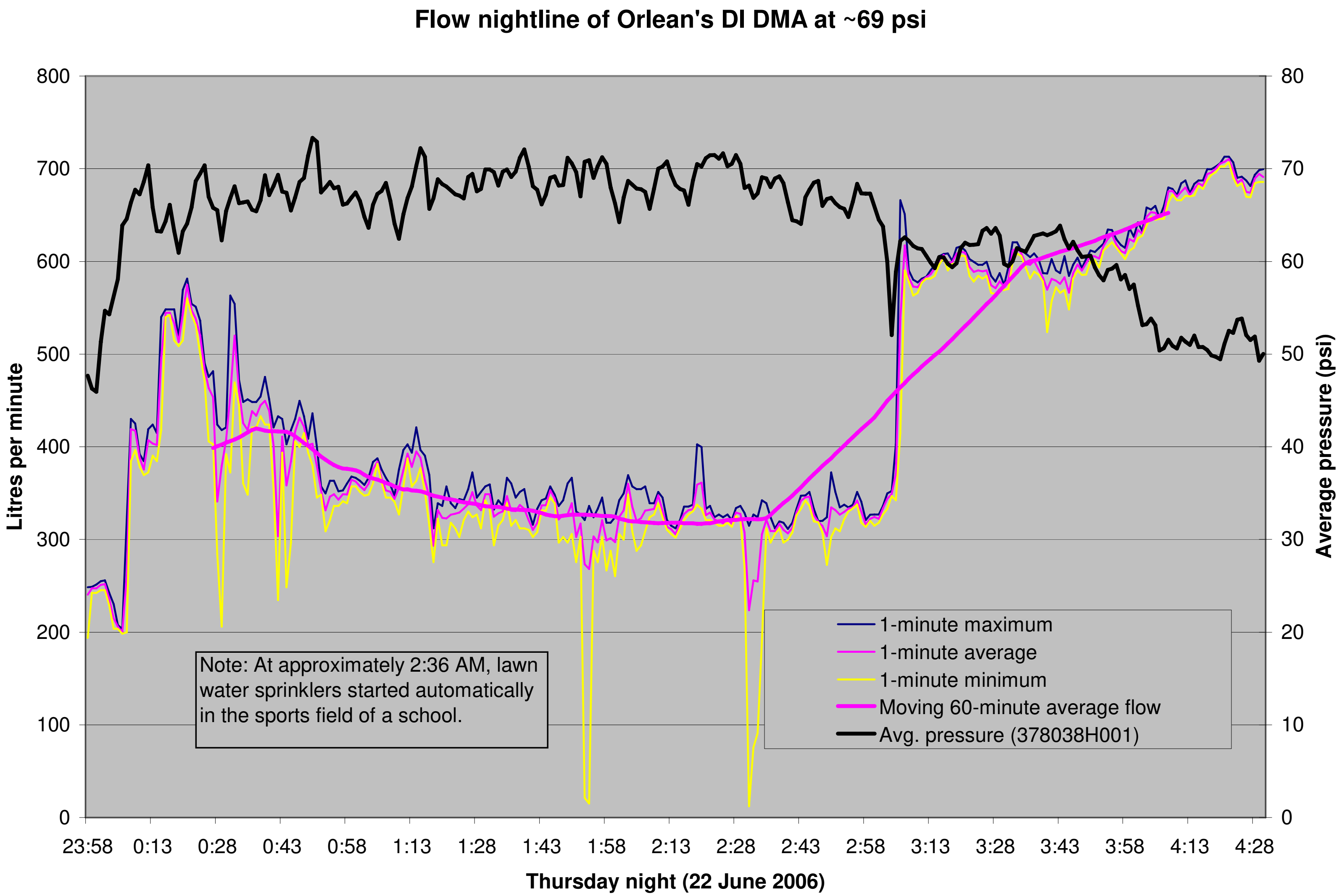
Flow nightline of Meadowland's DI DMA at ~44 psi

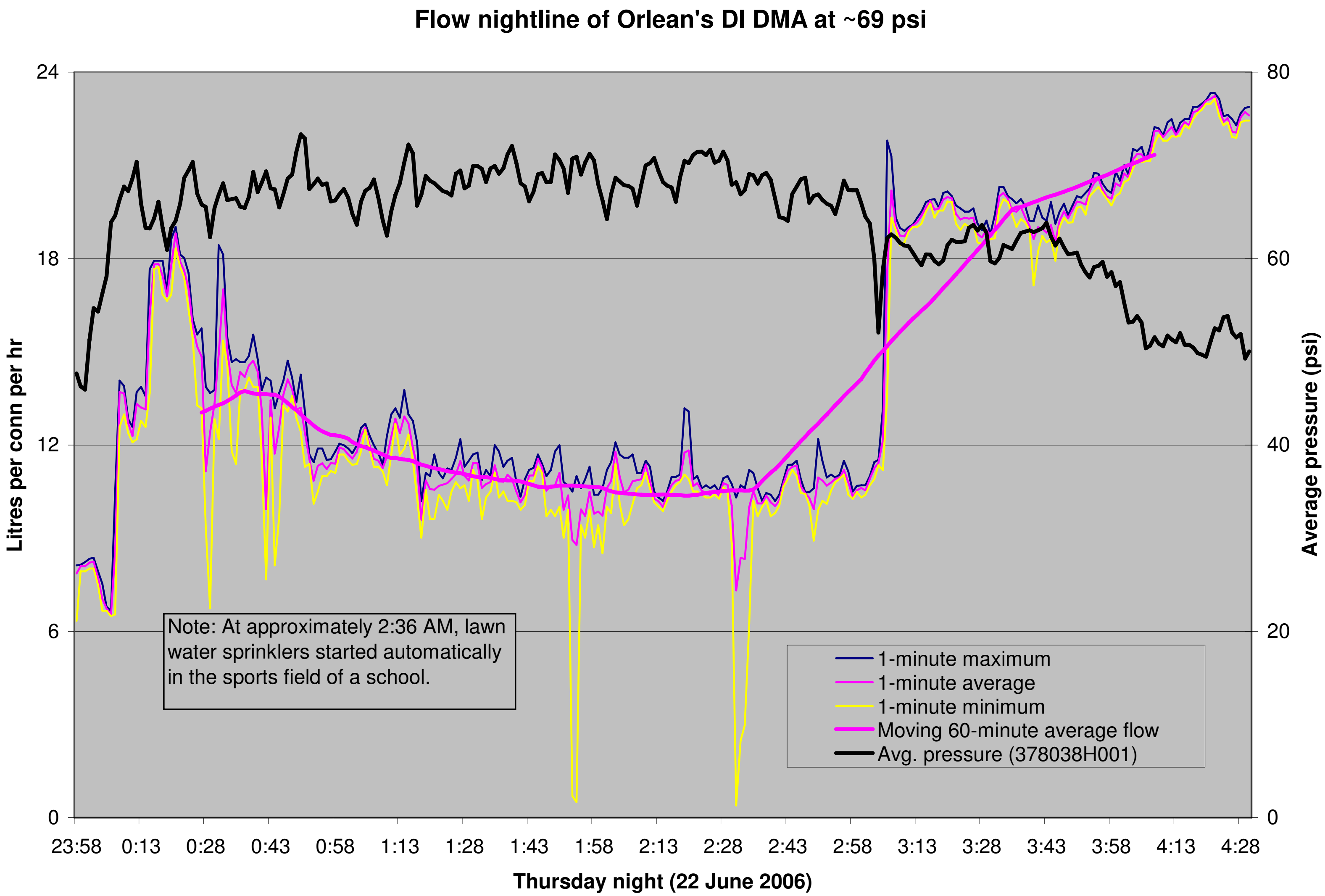




Flow nightline of Orlean's DI DMA at ~56 psi
Litres per minute
Pressure (psi)
Wednesday night (21 June 2006)
0
50
100
150
200
250
300
350
400
450
500
10
15
20
25
30
35
40
45
50
55
60
0:30
0:45
1:00
1:15
1:30
1:45
2:00
2:15
2:30
2:45
3:00
3:15
3:30
3:45
4:00
4:15
1-minute maximum
1-minute average
1-minute minimum
Moving 60-minute average flow
Avg. pressure (378038H001)

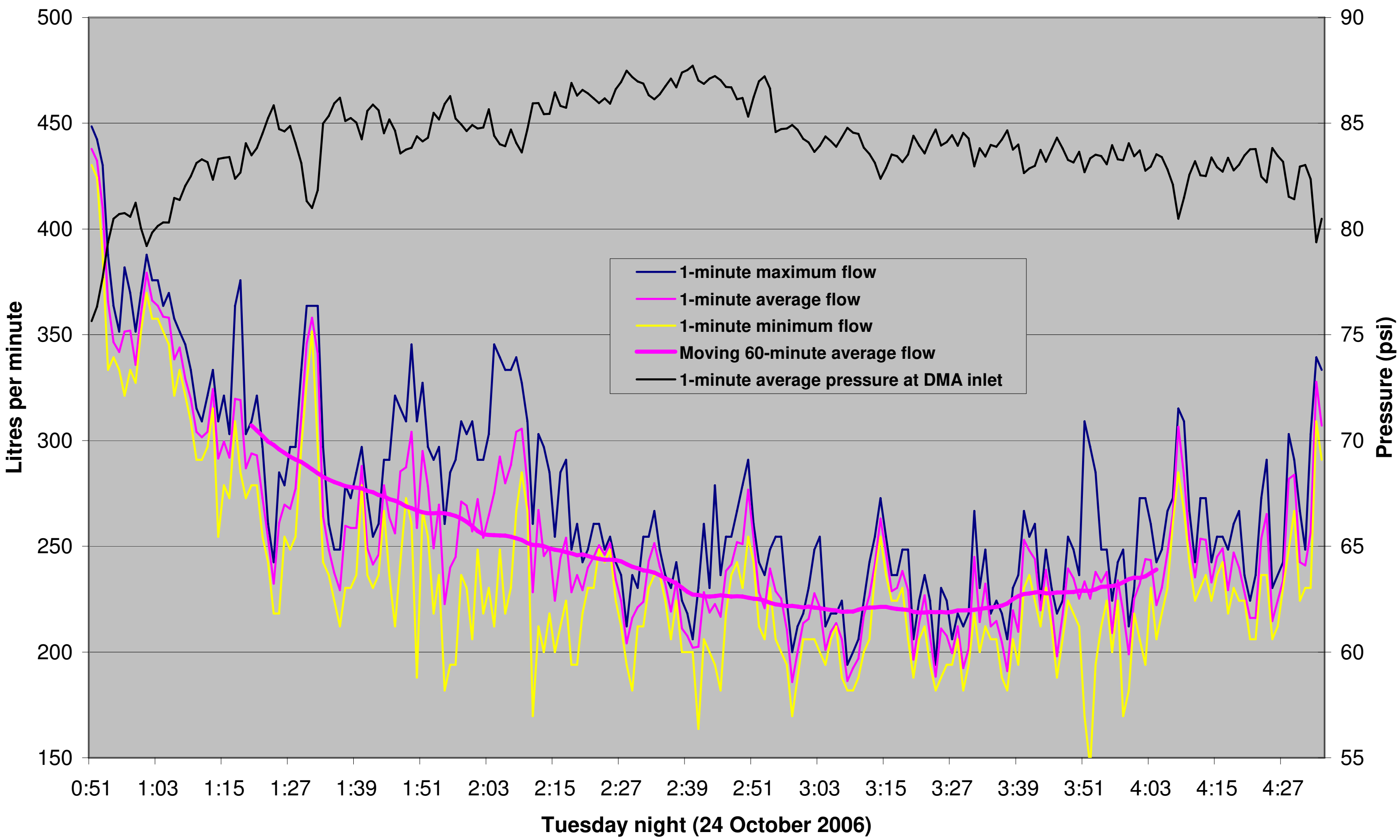

Flow nightline of Orlean's DI DMA at ~56 psi
Litres per conn per hr
Pressure (psi)
Wednesday night (21 June 2006)
0:30
0:45
1:00
1:15
1:30
1:45
2:00
2:15
2:30
2:45
3:00
3:15
3:30
3:45
4:00
4:15
0
2
4
6
8
10
12
14
16
20
25
30
35
40
45
50
55
60
1-minute maximum
1-minute average
1-minute minimum
Moving 60-minute average flow
Avg. pressure (378038H001)

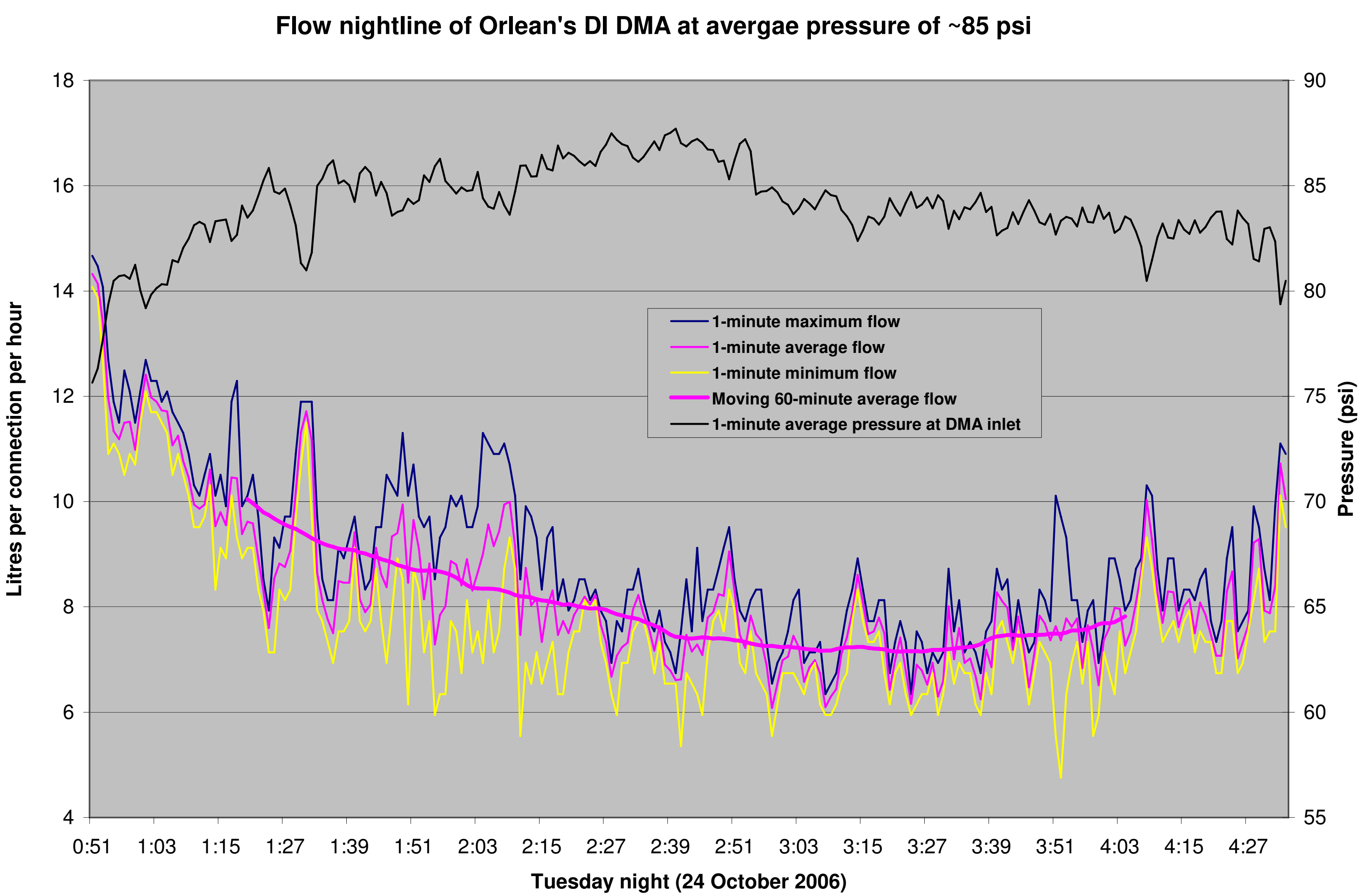

Flow nightline of Orlean's DI DMA at ~69 psi
Litres per minute
800
700
600
500
400
300
200
100
0
Average pressure (psi)
80
70
60
50
40
30
20
10
0
23:58
0:13
0:28
0:43
0:58
1:13
1:28
1:43
1:58
2:13
2:28
2:43
2:58
3:13
3:28
3:43
3:58
4:13
4:28
Thursday night (22 June 2006)
Note: At approximately 2:36 AM, lawn water sprinklers started automatically in the sports field of a school.
1-minute maximum
1-minute average
1-minute minimum
Moving 60-minute average flow
Avg. pressure (378038H001)



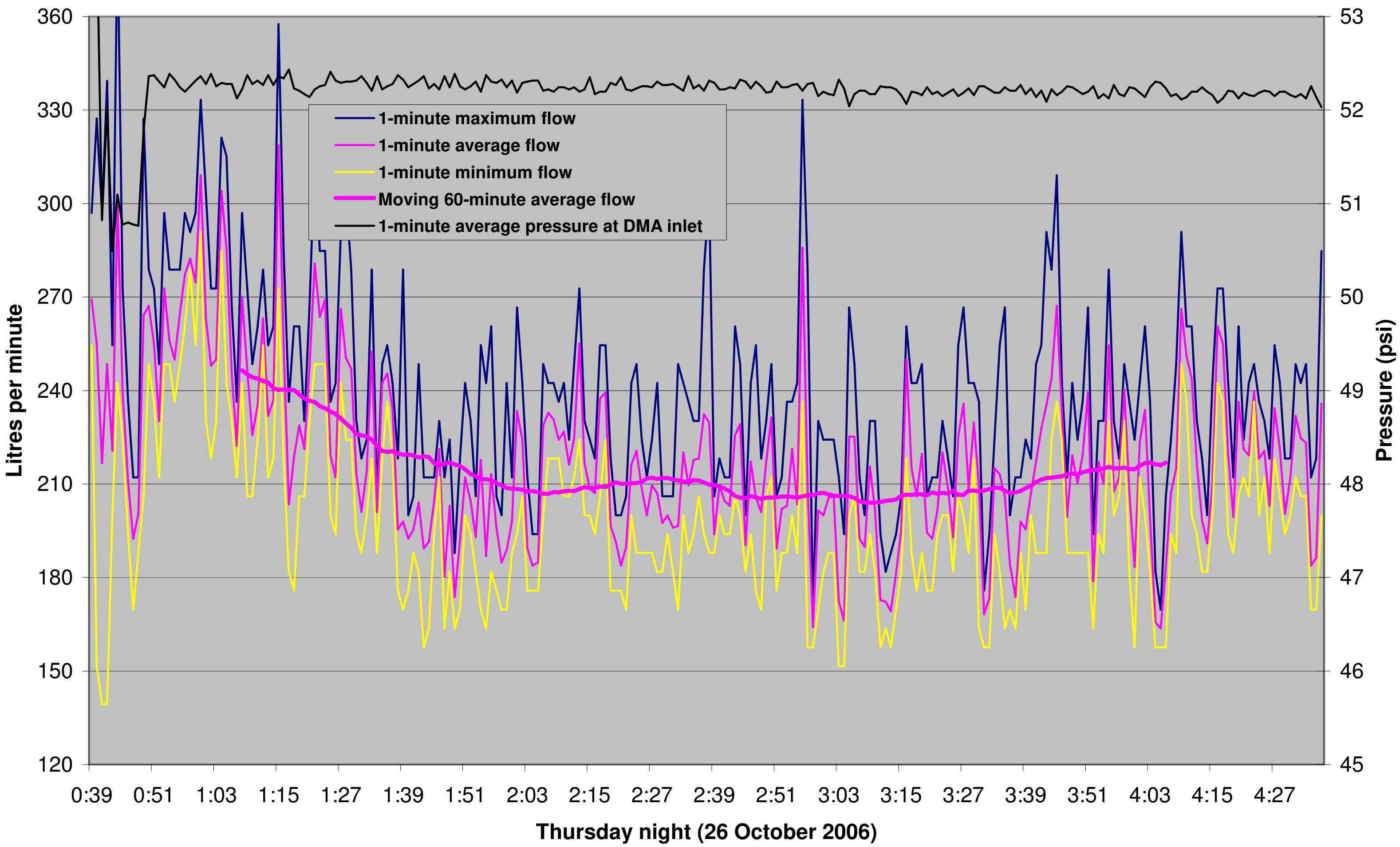
Flow nightline of Orlean's DI DMA at ~69 psi
Litres per conn per hr
Average pressure (psi)
Thursday night (22 June 2006)
0
6
12
18
24
20
40
60
80
23:58
0:13
0:28
0:43
0:58
1:13
1:28
1:43
1:58
2:13
2:28
2:43
2:58
3:13
3:28
3:43
3:58
4:13
4:28
Note: At approximately 2:36 AM, lawn water sprinklers started automatically in the sports field of a school.
1-minute maximum
1-minute average
1-minute minimum
Moving 60-minute average flow
Avg. pressure (378038H001)

Flow nightline of Orlean's DI DMA at avergae pressure of ~85 psi

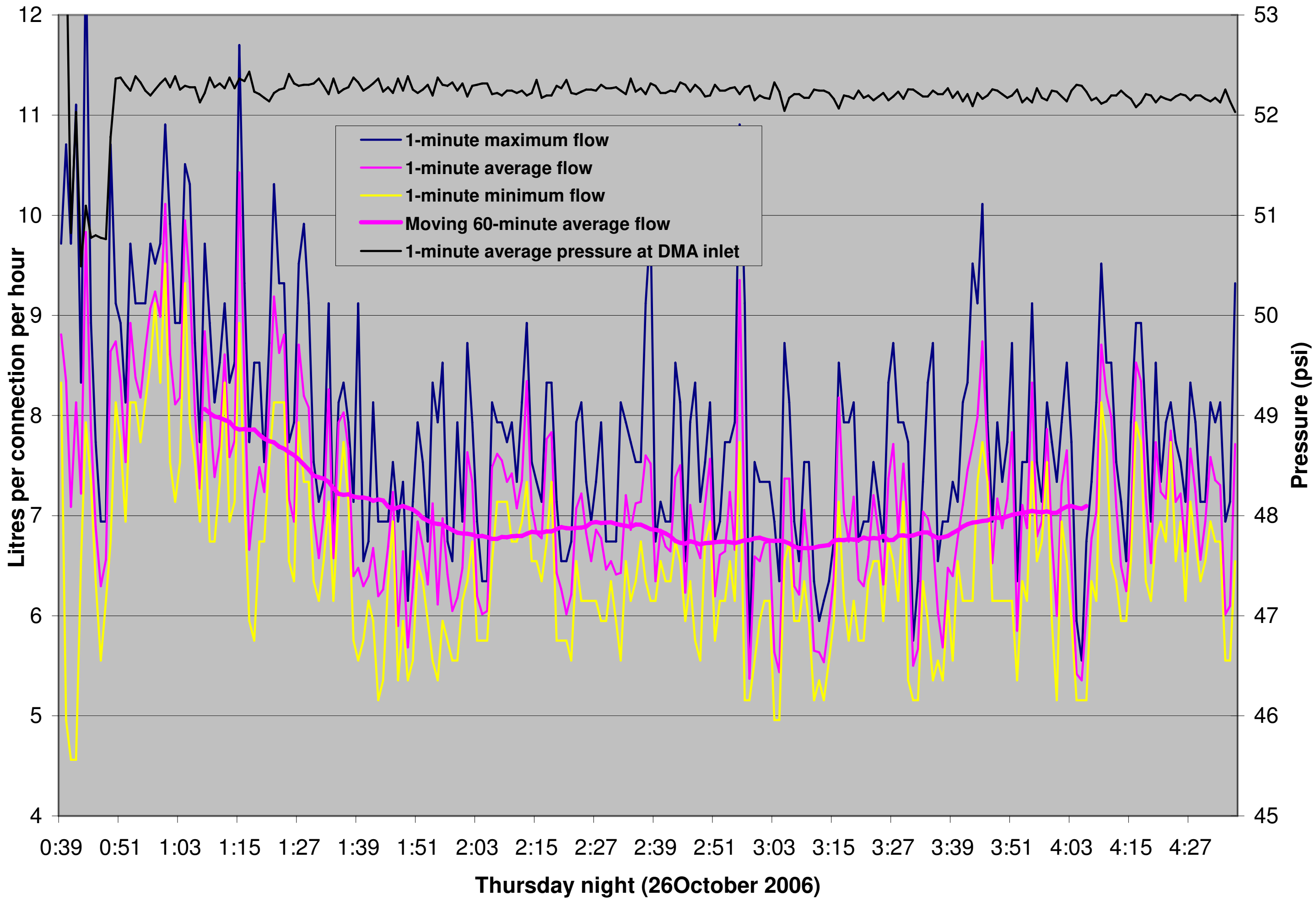



Flow nightline of Orlean's DI DMA at avergae pressure of ~85 psi
Litres per connection per hour
Pressure (psi)
Tuesday night (24 October 2006)
1-minute maximum flow
1-minute average flow
1-minute minimum flow
Moving 60-minute average flow
1-minute average pressure at DMA inlet
18
16
14
12
10
8
6
4
90
85
80
75
70
65
60
55
0:51
1:03
1:15
1:27
1:39
1:51
2:03
2:15
2:27
2:39
2:51
3:03
3:15
3:27
3:39
3:51
4:03
4:15
4:27

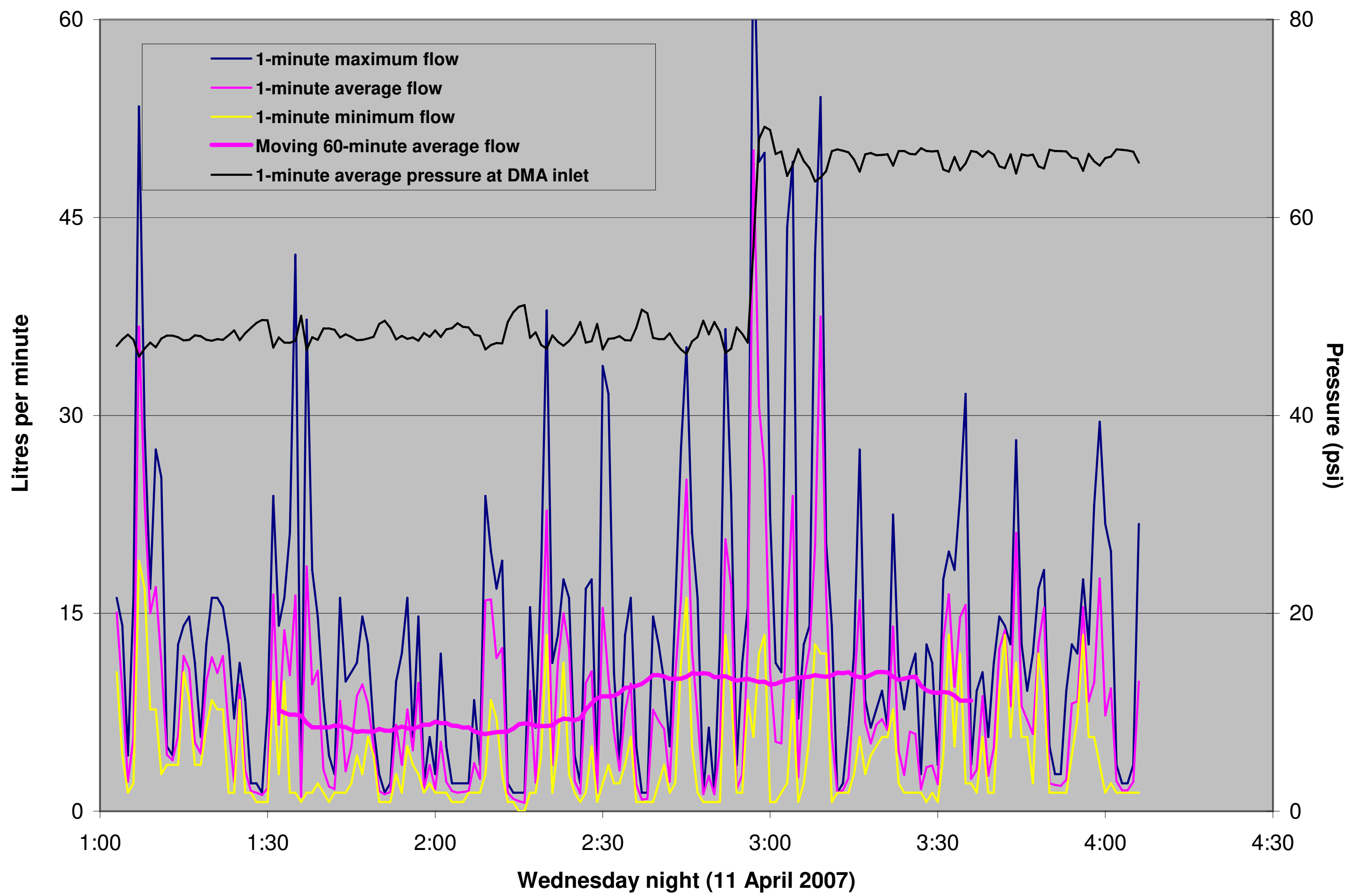
Flow nightline of Orlean's DI DMA at reduced pressureof ~52 psi



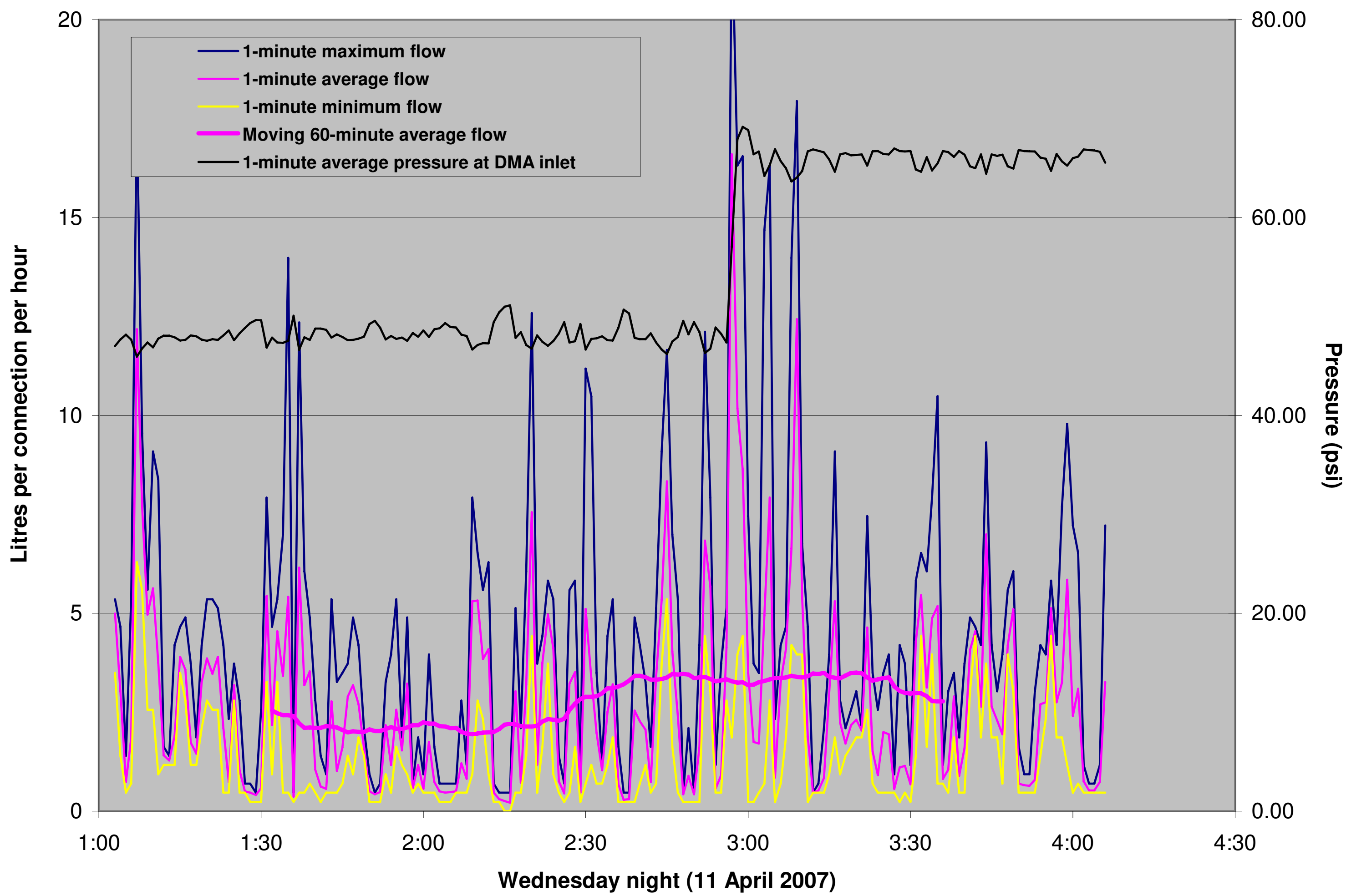
Flow nightline of Orlean's DI DMA at reduced pressureof ~52 psi



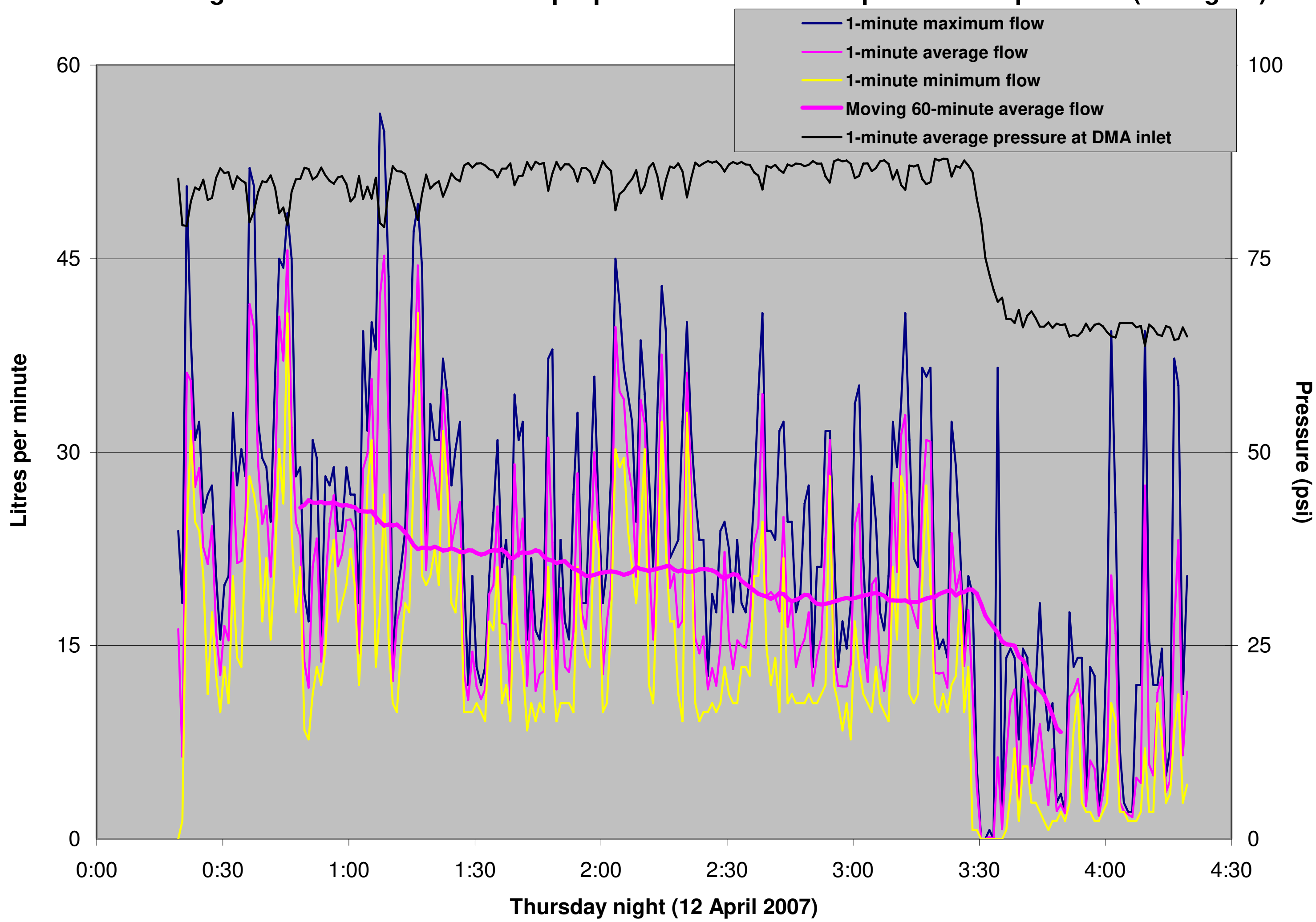
Flow nightline of Meadowlands' CI purple sub-DMA at 51/68 psi average pressure (LoLog LL)

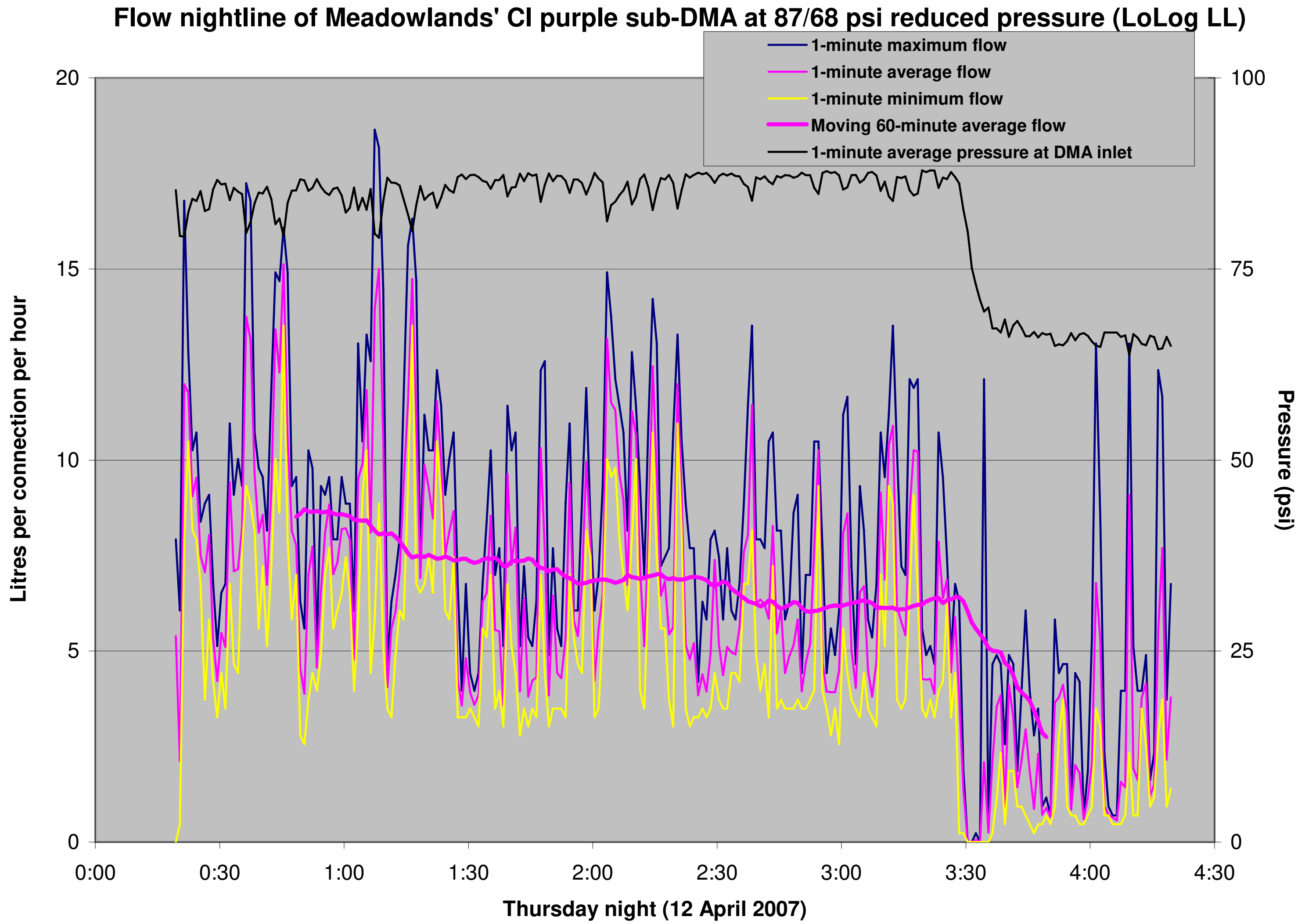


Flow nightline of Meadowlands' CI purple sub-DMA at 51/68 psi average pressure (LoLog LL)



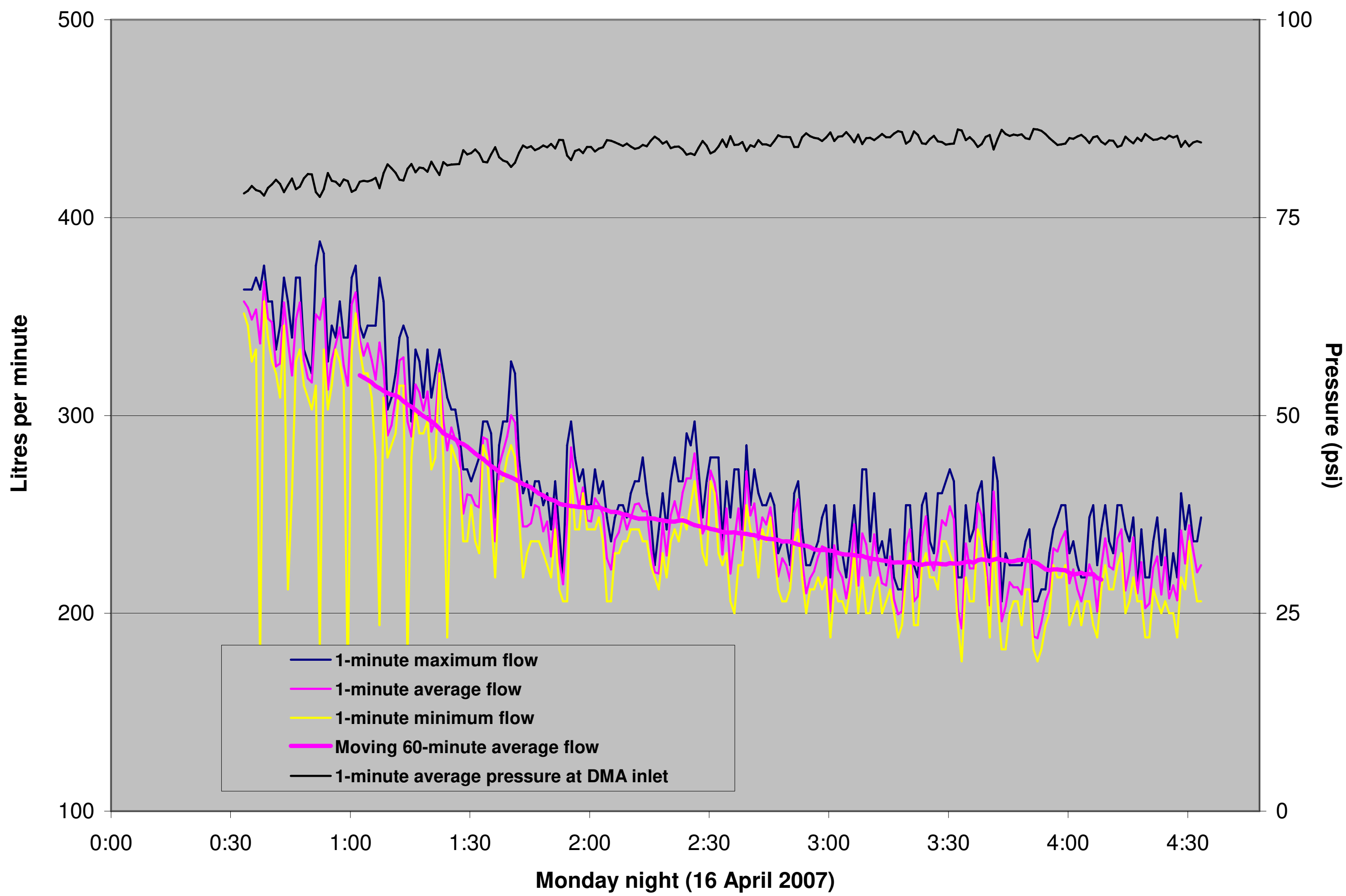
Flow nightline of Meadowlands' CI purple sub-DMA at 87/68 psi reduced pressure (LoLog LL)

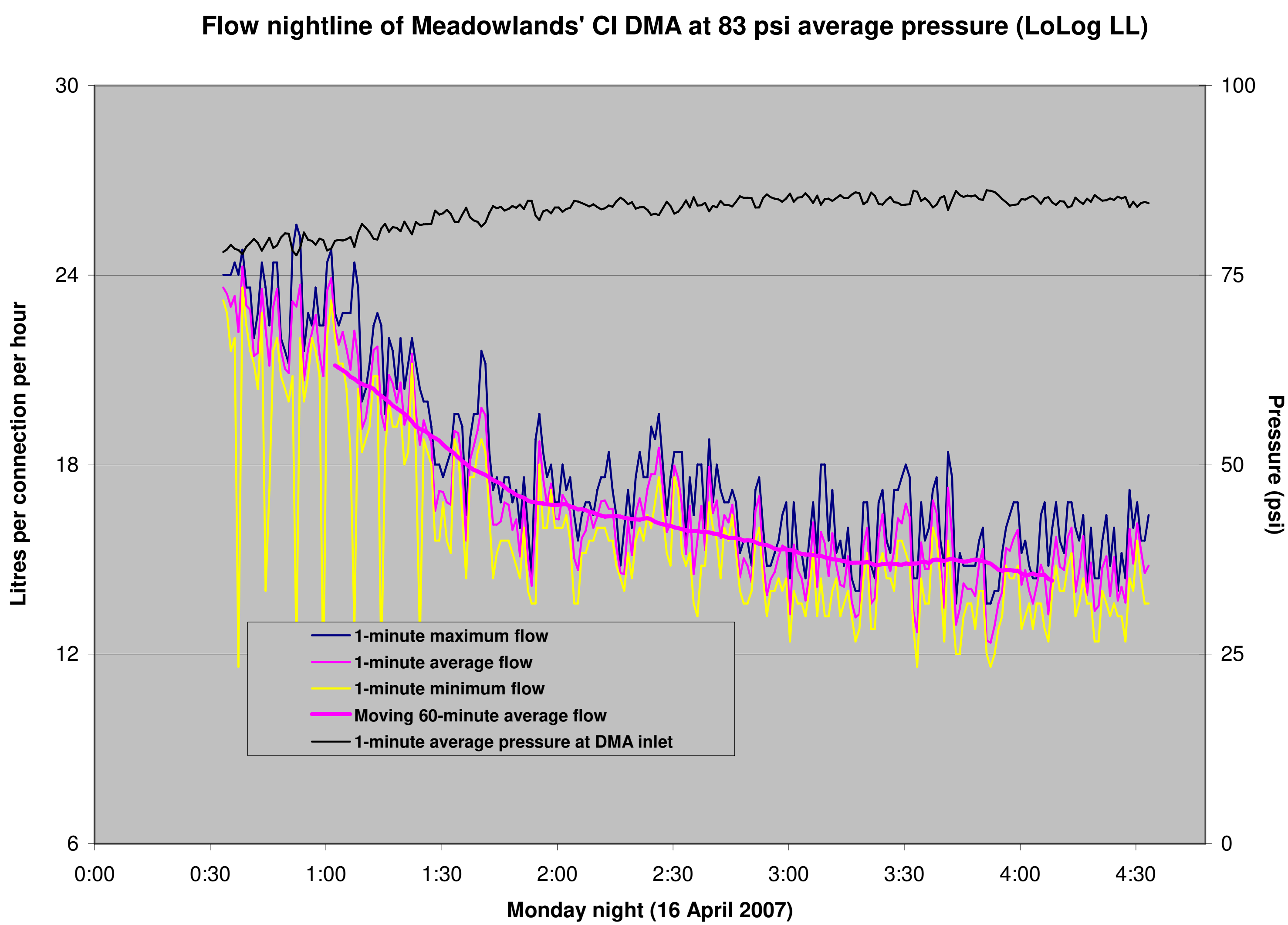




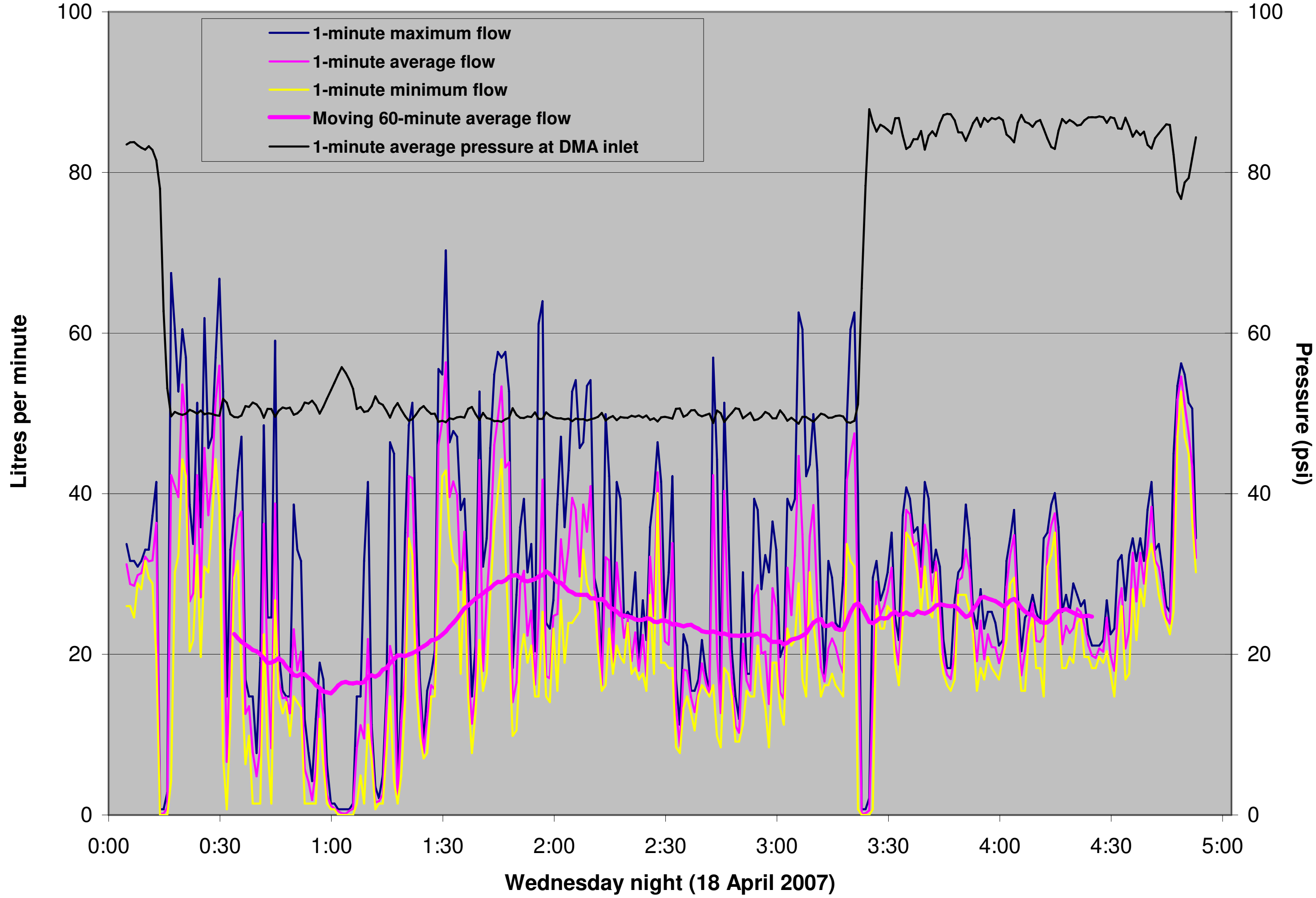
Flow nightline of Meadowlands' CI purple sub-DMA at 87/68 psi reduced pressure (LoLog LL)
1-minute maximum flow
1-minute average flow
1-minute minimum flow
Moving 60-minute average flow
1-minute average pressure at DMA inlet
Litres per connection per hour
Pressure (psi)
20
15
10
5
0
100
75
50
25
0
0:00
0:30
1:00
1:30
2:00
2:30
3:00
3:30
4:00
4:30
Thursday night (12 April 2007)

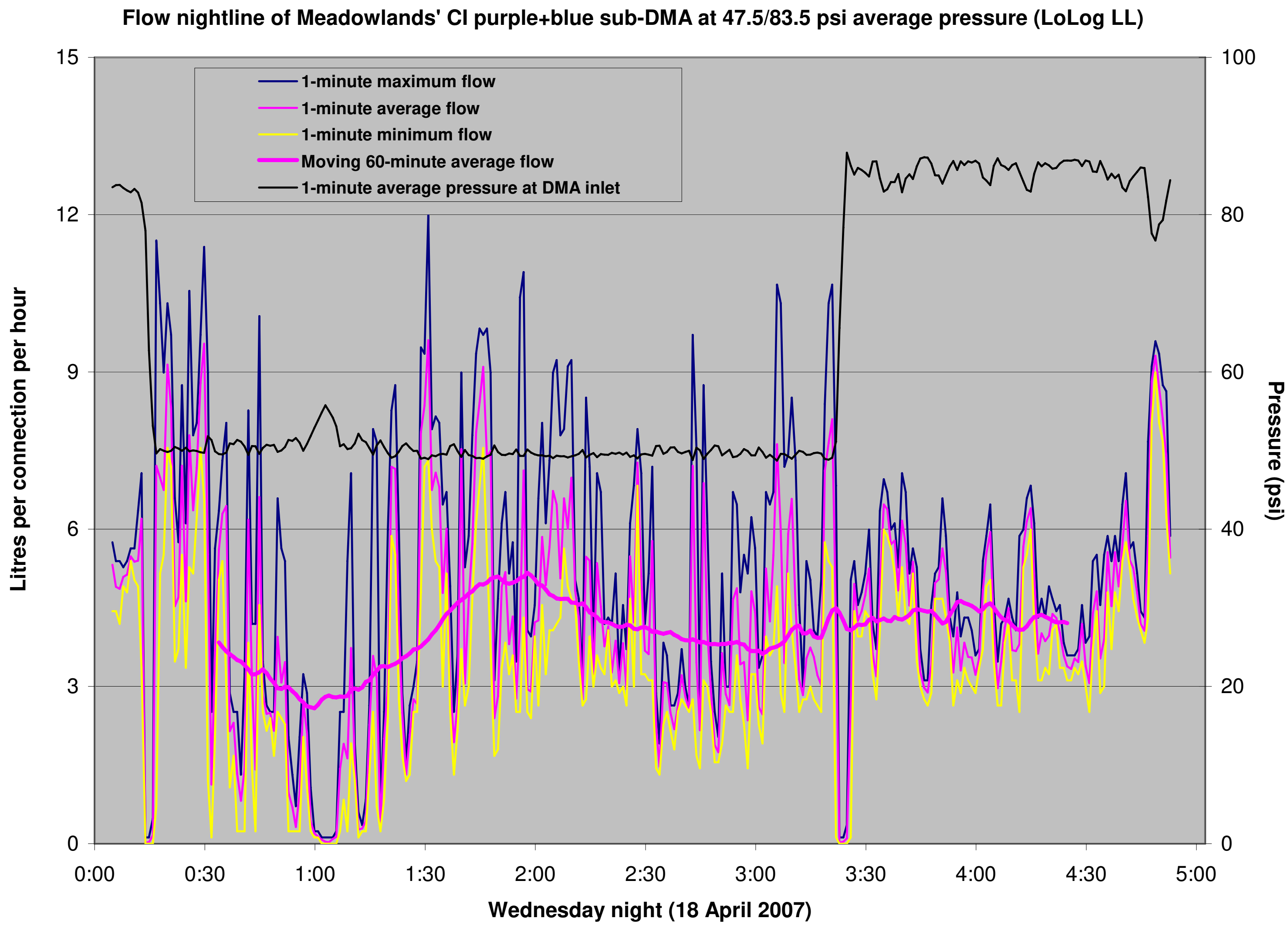
Flow nightline of Meadowlands' CI DMA at 83 psi average pressure (LoLog LL)

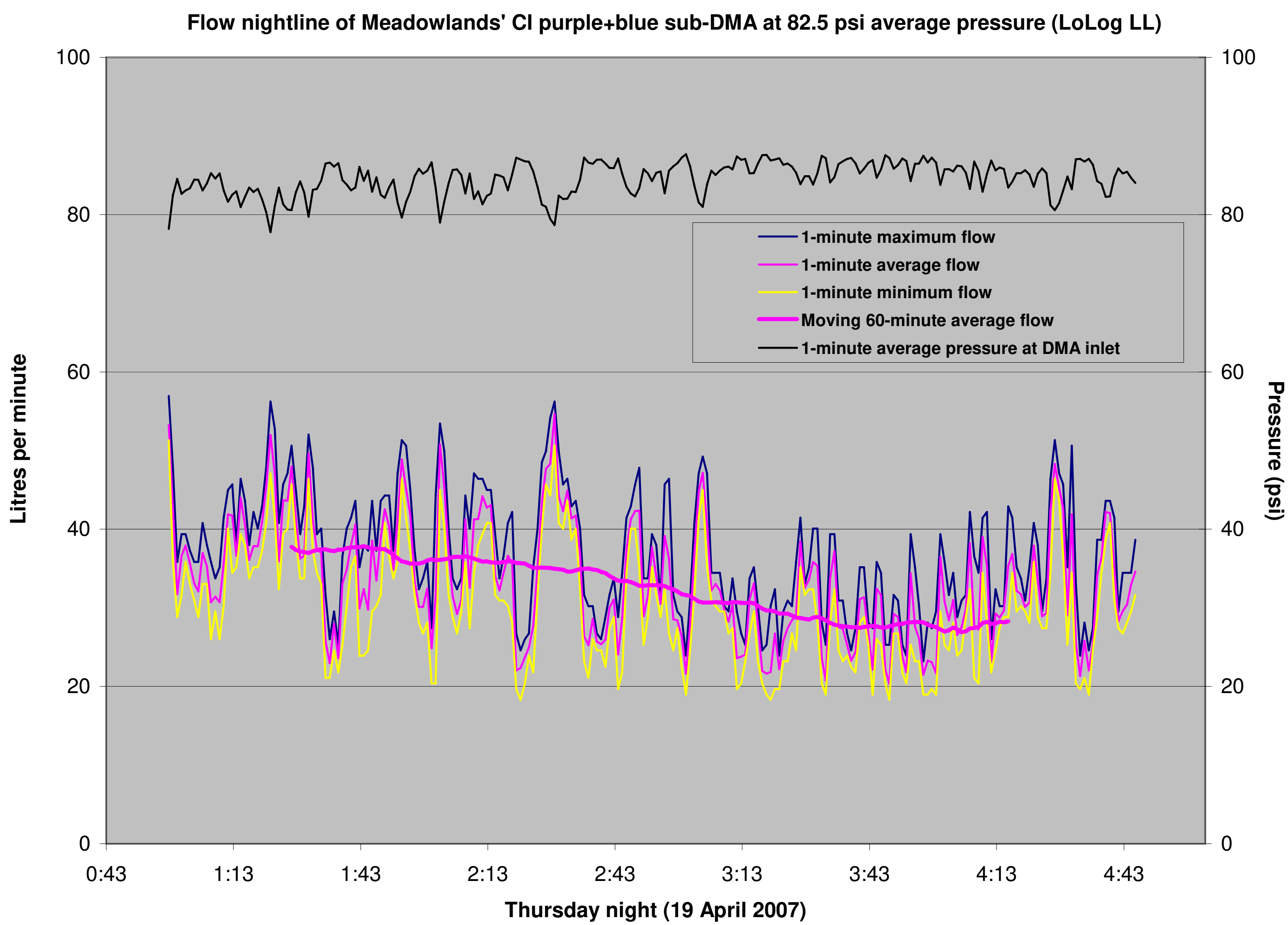



Flow nightline of Meadowlands' CI DMA at 83 psi average pressure (LoLog LL)
Litres per connection per hour
Pressure (psi)
30
24
18
12
6
100
75
50
25
0
0:00
0:30
1:00
1:30
2:00
2:30
3:00
3:30
4:00
4:30
Monday night (16 April 2007)
1-minute maximum flow
1-minute average flow
1-minute minimum flow
Moving 60-minute average flow
1-minute average pressure at DMA inlet

Flow nightline of Meadowlands' CI purple+blue sub-DMA at 47.5/83.5 psi average pressure (LoLog LL)

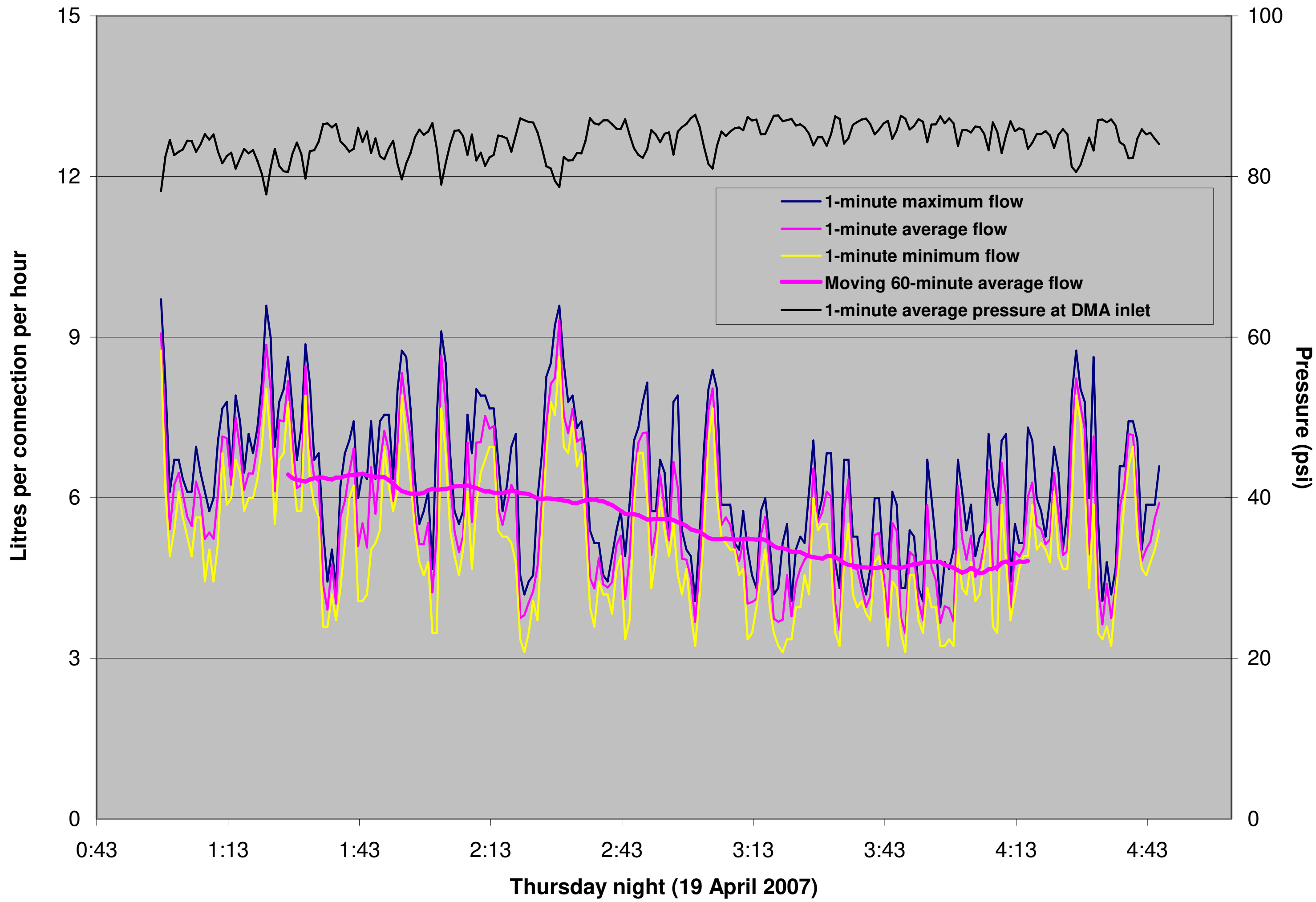



Flow nightline of Meadowlands' CI purple+blue sub-DMA at 47.5/83.5 psi average pressure (LoLog LL)
1-minute maximum flow
1-minute average flow
1-minute minimum flow
Moving 60-minute average flow
1-minute average pressure at DMA inlet
Litres per connection per hour
Pressure (psi)
Wednesday night (18 April 2007)
0
3
6
9
12
15
0
20
40
60
80
100
0:00
0:30
1:00
1:30
2:00
2:30
3:00
3:30
4:00
4:30
5:00

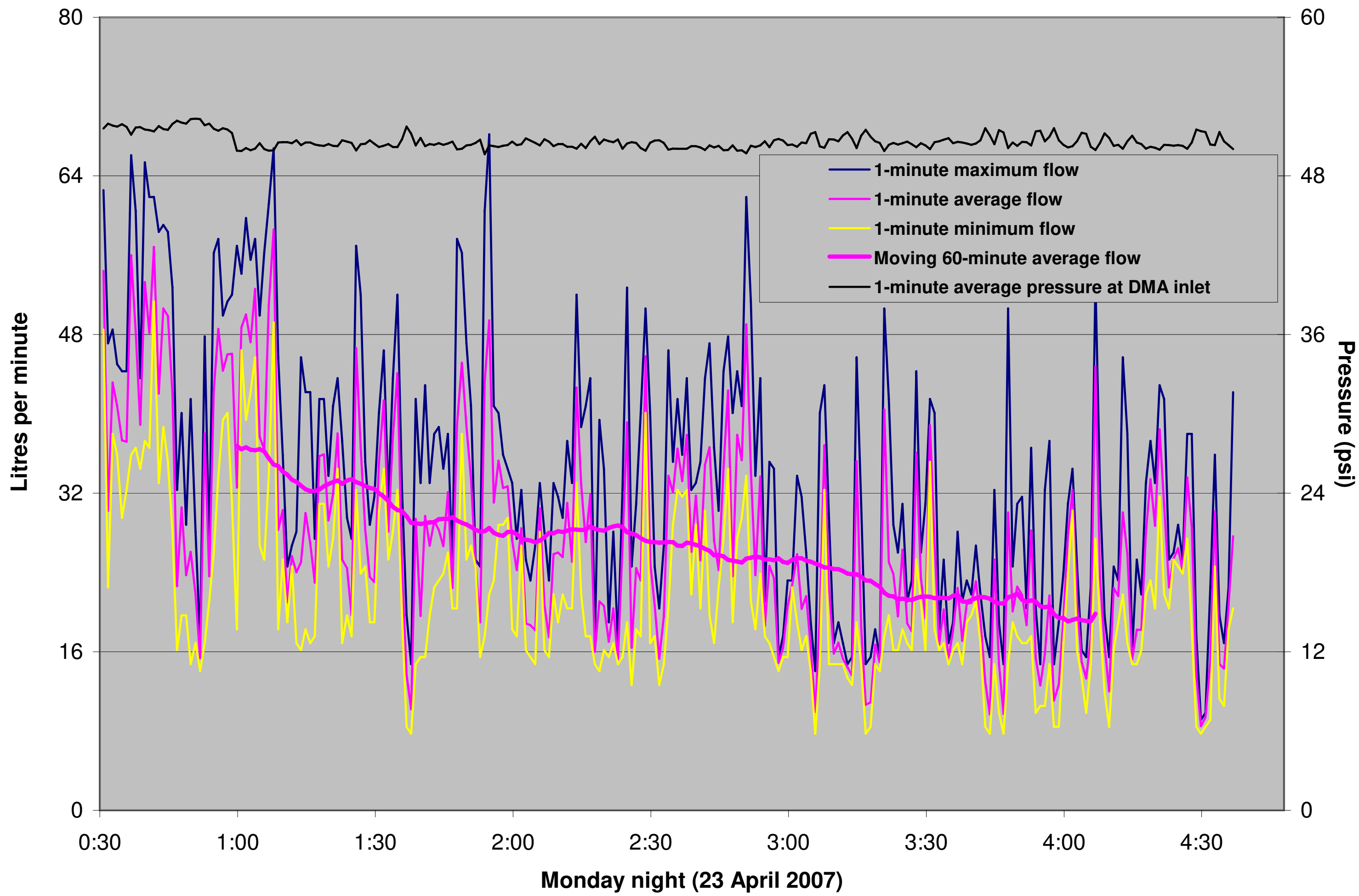


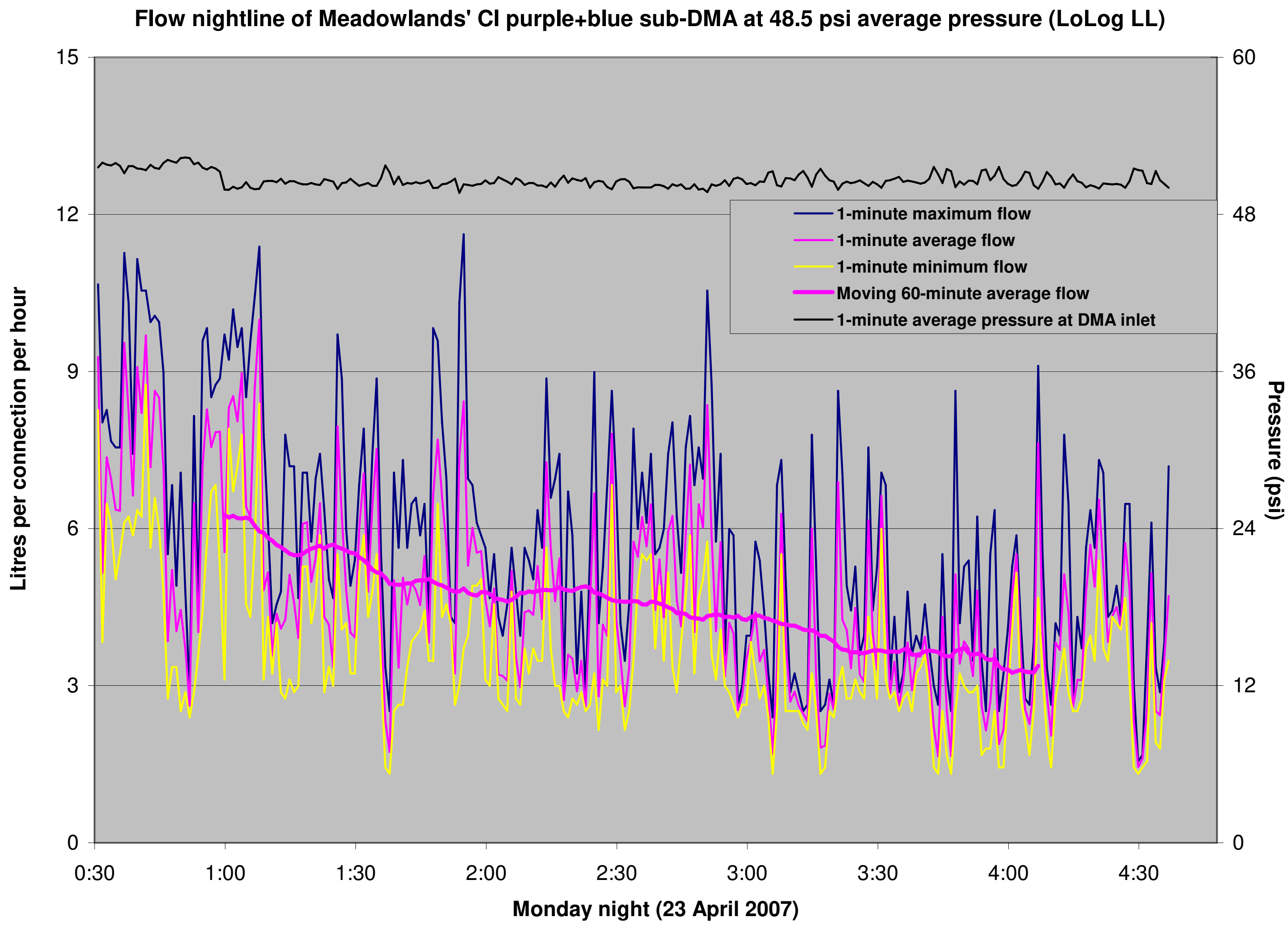
Flow nightline of Meadowlands' CI purple+blue sub-DMA at 82.5 psi average pressure (LoLog LL)
Litres per minute
Pressure (psi)
0
20
40
60
80
100
0:43
1:13
1:43
2:13
2:43
3:13
3:43
4:13
4:43
Thursday night (19 April 2007)
1-minute maximum flow
1-minute average flow
1-minute minimum flow
Moving 60-minute average flow
1-minute average pressure at DMA inlet

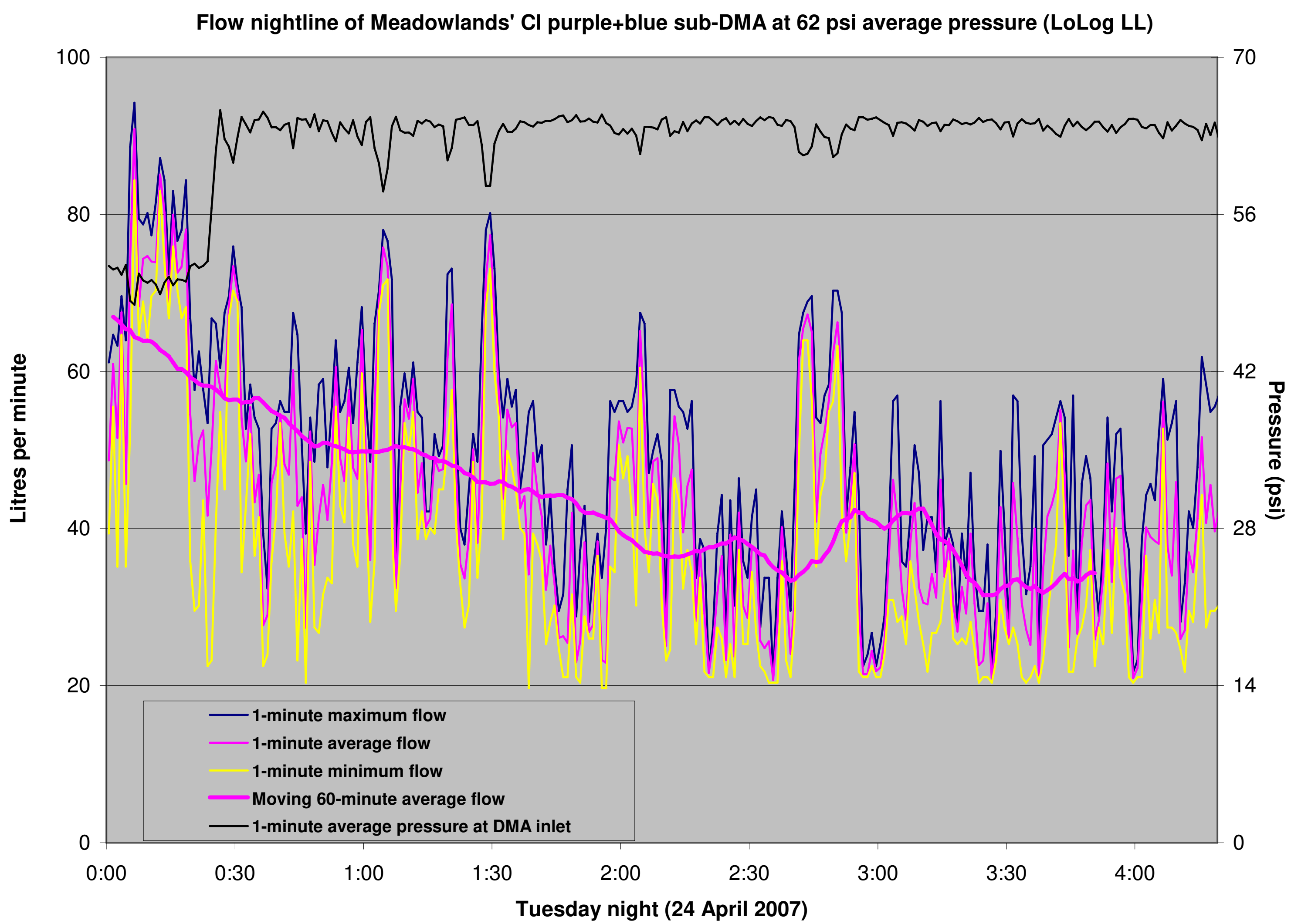
Flow nightline of Meadowlands' CI purple+blue sub-DMA at 82.5 psi average pressure (LoLog LL)



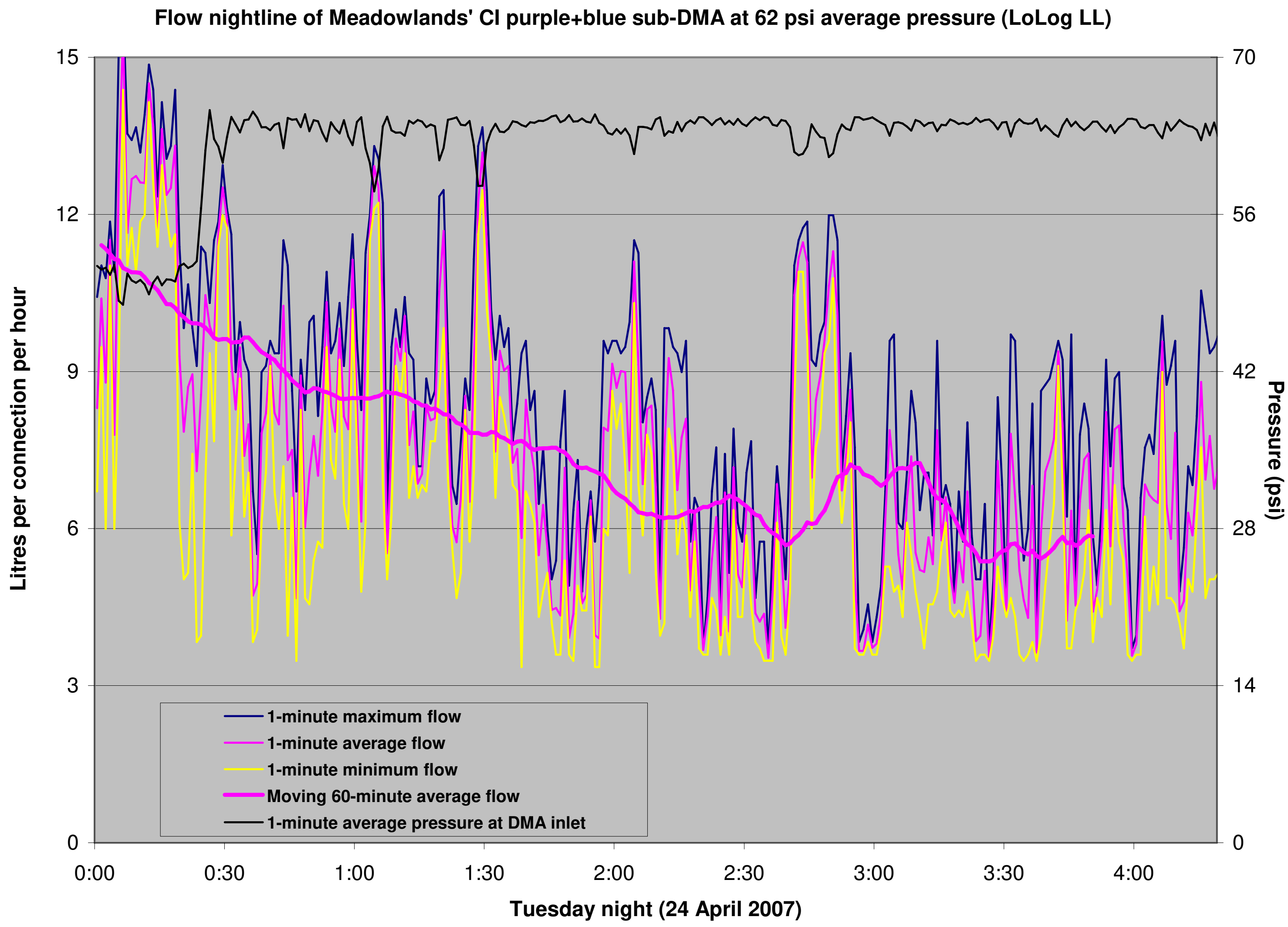
Flow nightline of Meadowlands Cl purple+blue sub-DMA at 48.5 psi average pressure (LoLog LL)

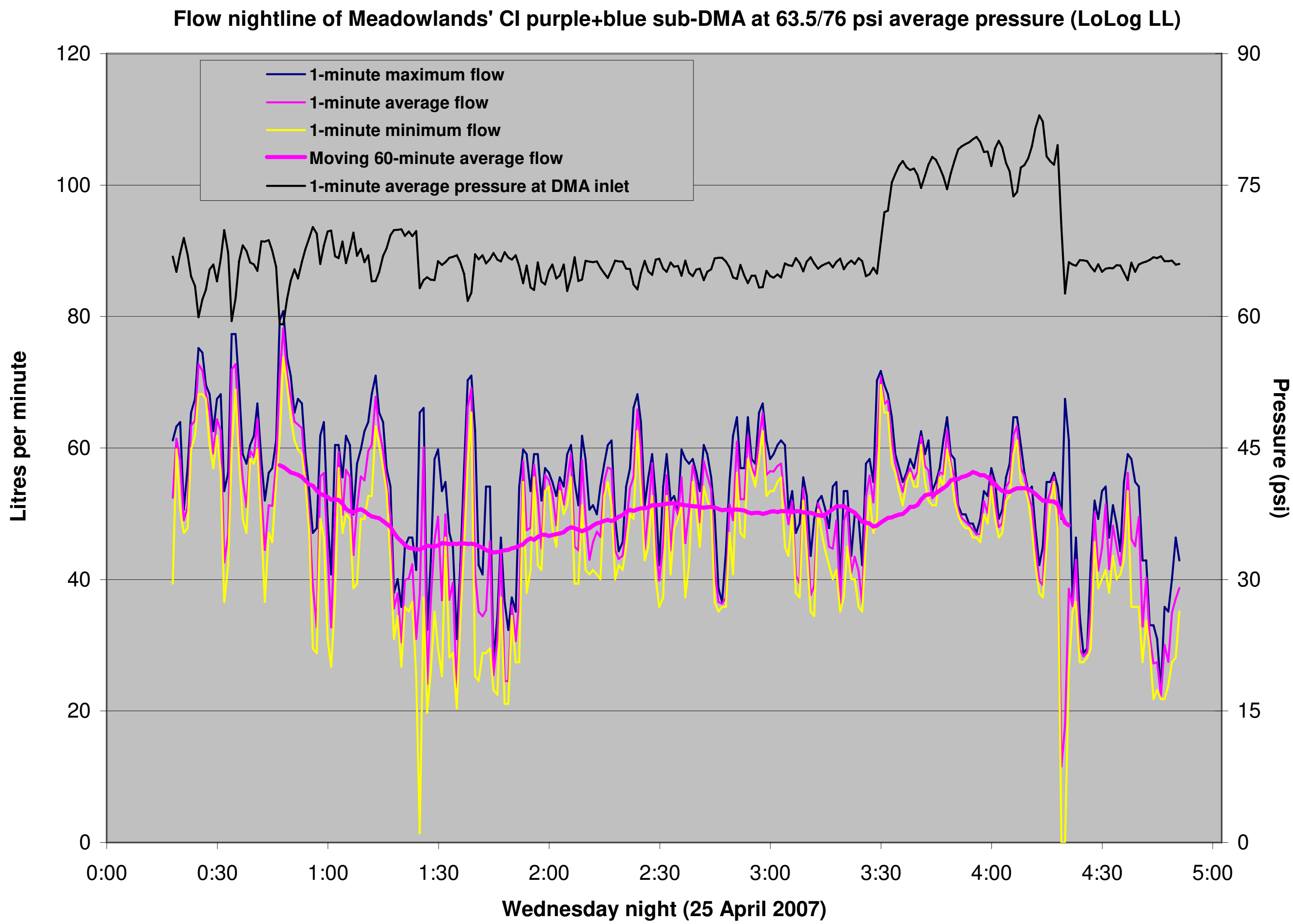



Flow nightline of Meadowlands' CI purple+blue sub-DMA at 48.5 psi average pressure (LoLog LL)
Litres per connection per hour
Pressure (psi)
Monday night (23 April 2007)
0:30
1:00
1:30
2:00
2:30
3:00
3:30
4:00
4:30
0
3
6
9
12
15
0
12
24
36
48
60
1-minute maximum flow
1-minute average flow
1-minute minimum flow
Moving 60-minute average flow
1-minute average pressure at DMA inlet

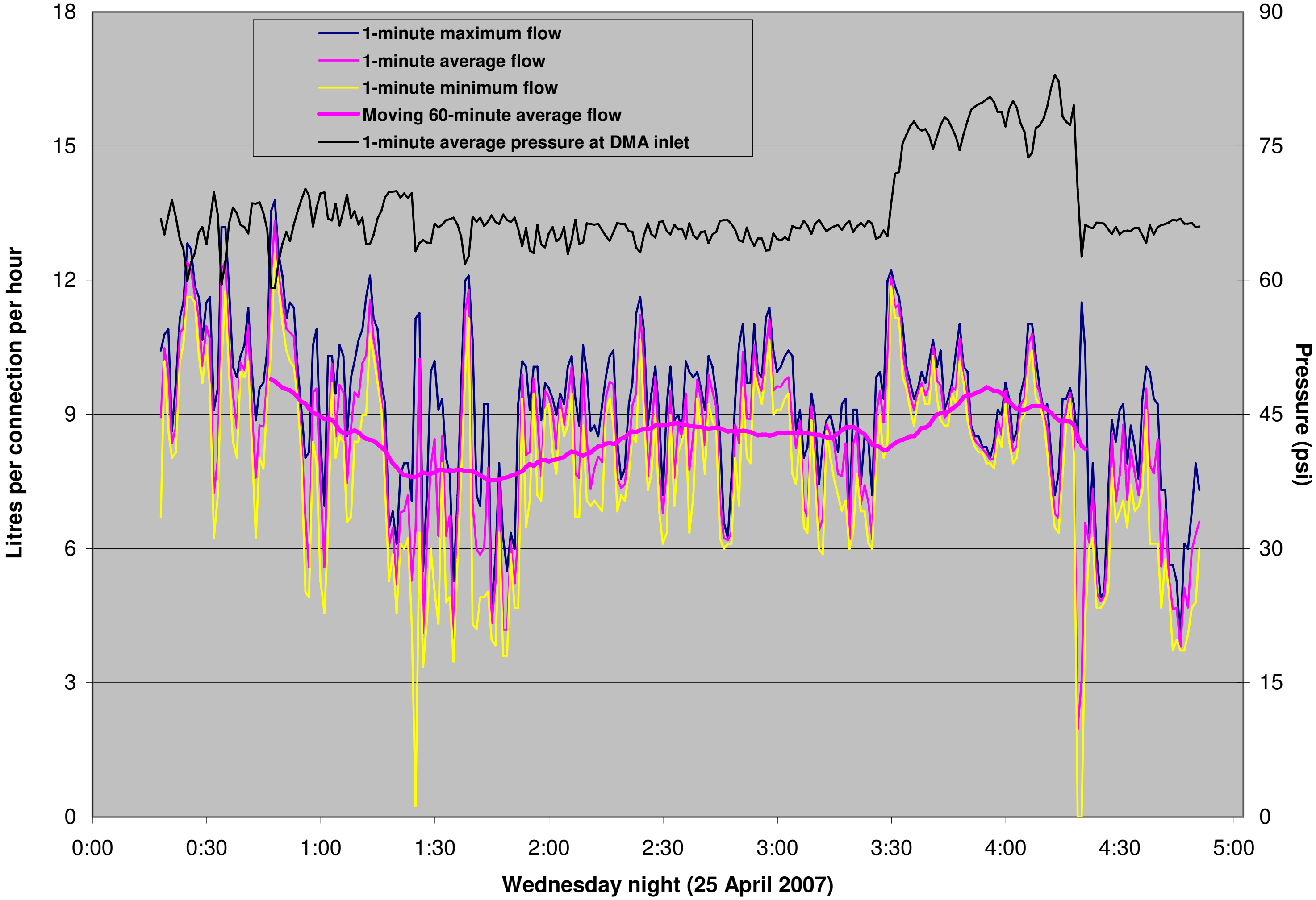


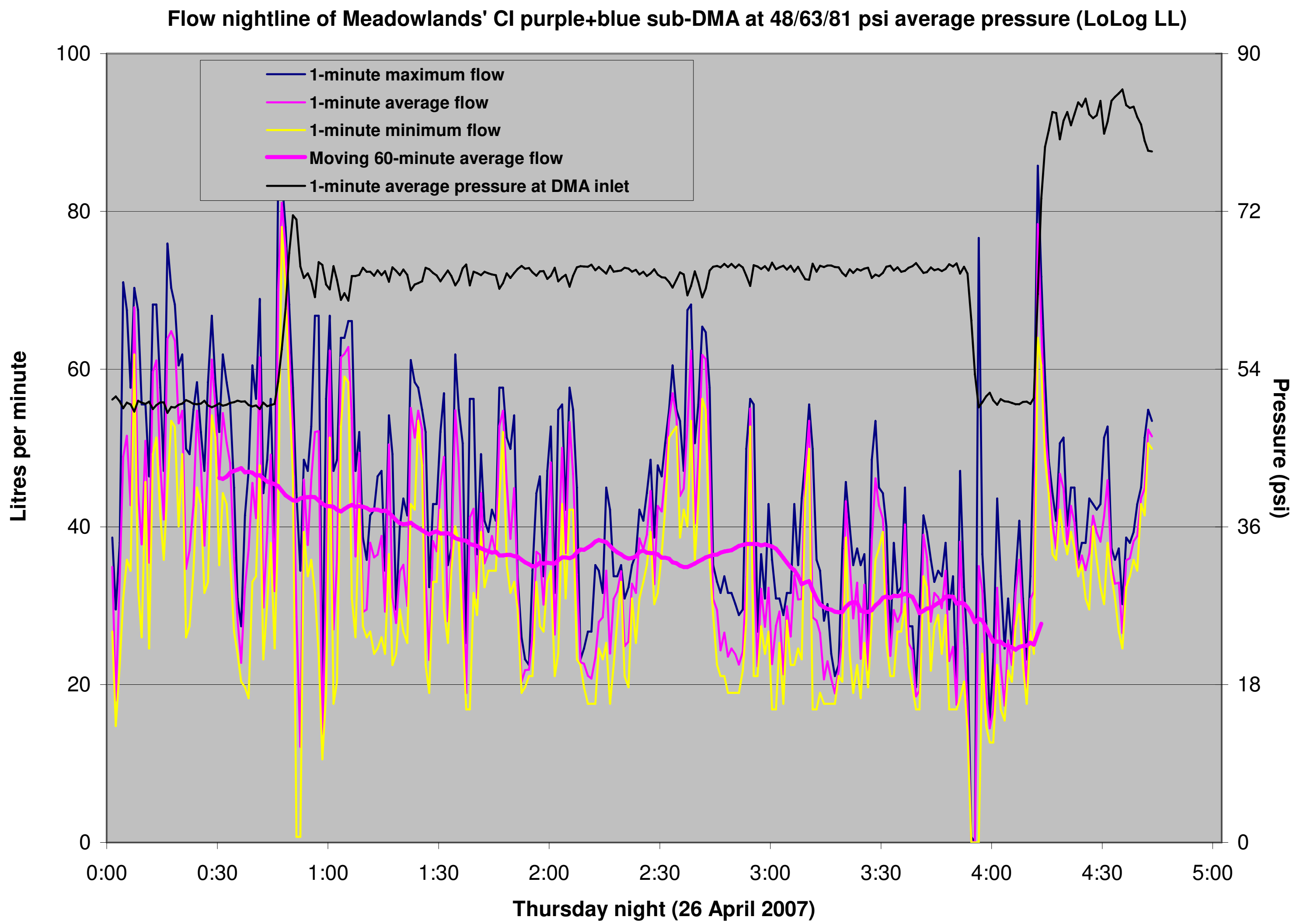
Flow nightline of Meadowlands' CI purple+blue sub-DMA at 62 psi average pressure (LoLog LL)
Litres per minute
Pressure (psi)
100
80
60
40
20
0
70
56
42
28
14
0
0:00
0:30
1:00
1:30
2:00
2:30
3:00
3:30
4:00
Tuesday night (24 April 2007)
1-minute maximum flow
1-minute average flow
1-minute minimum flow
Moving 60-minute average flow
1-minute average pressure at DMA inlet

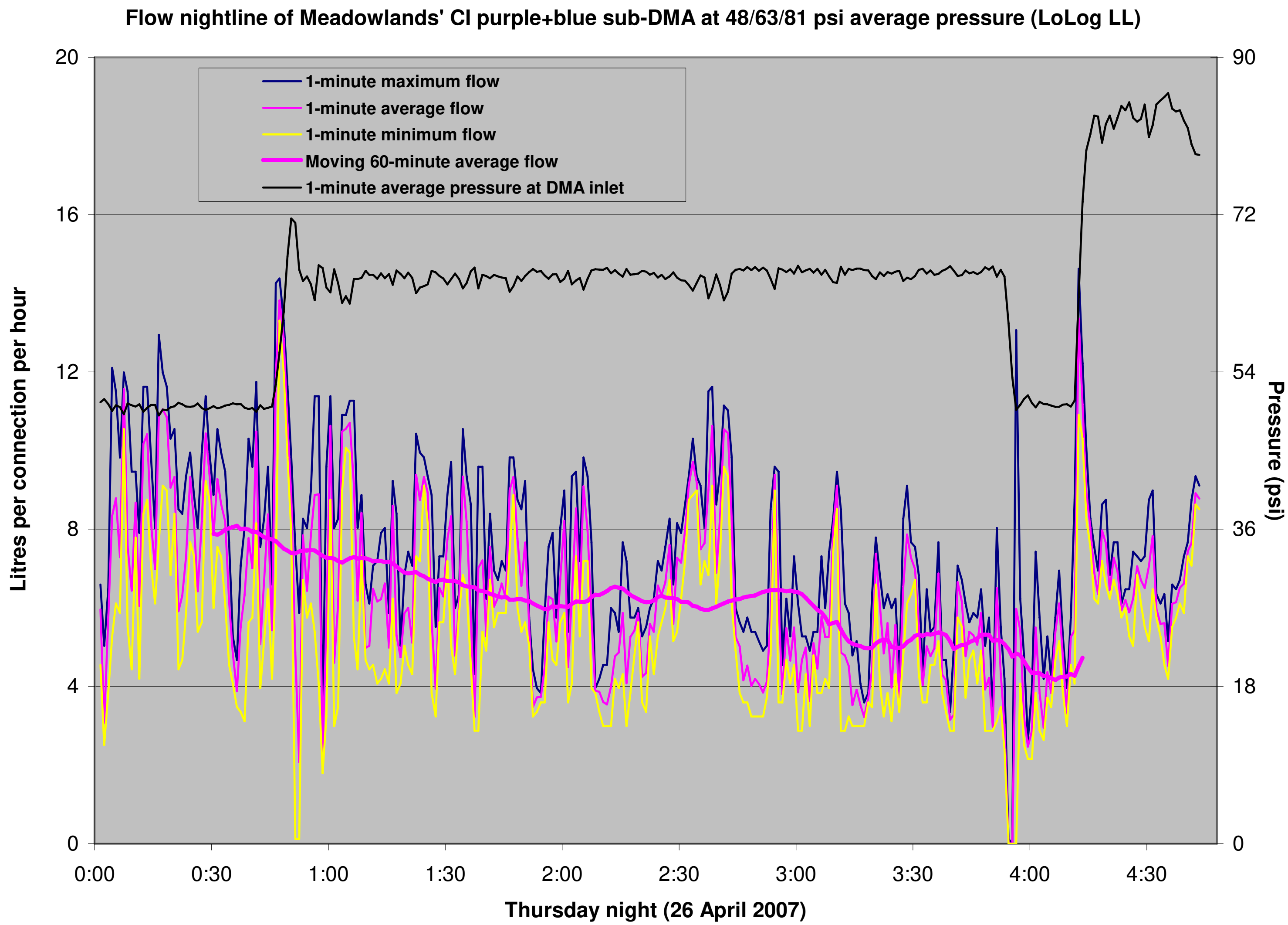

Flow nightline of Meadowlands' CI purple+blue sub-DMA at 62 psi average pressure (LoLog LL)
Litres per connection per hour
Pressure (psi)
15
12
9
6
3
0
70
56
42
28
14
0
1-minute maximum flow
1-minute average flow
1-minute minimum flow
Moving 60-minute average flow
1-minute average pressure at DMA inlet
0:00
0:30
1:00
1:30
2:00
2:30
3:00
3:30
4:00
Tuesday night (24 April 2007)

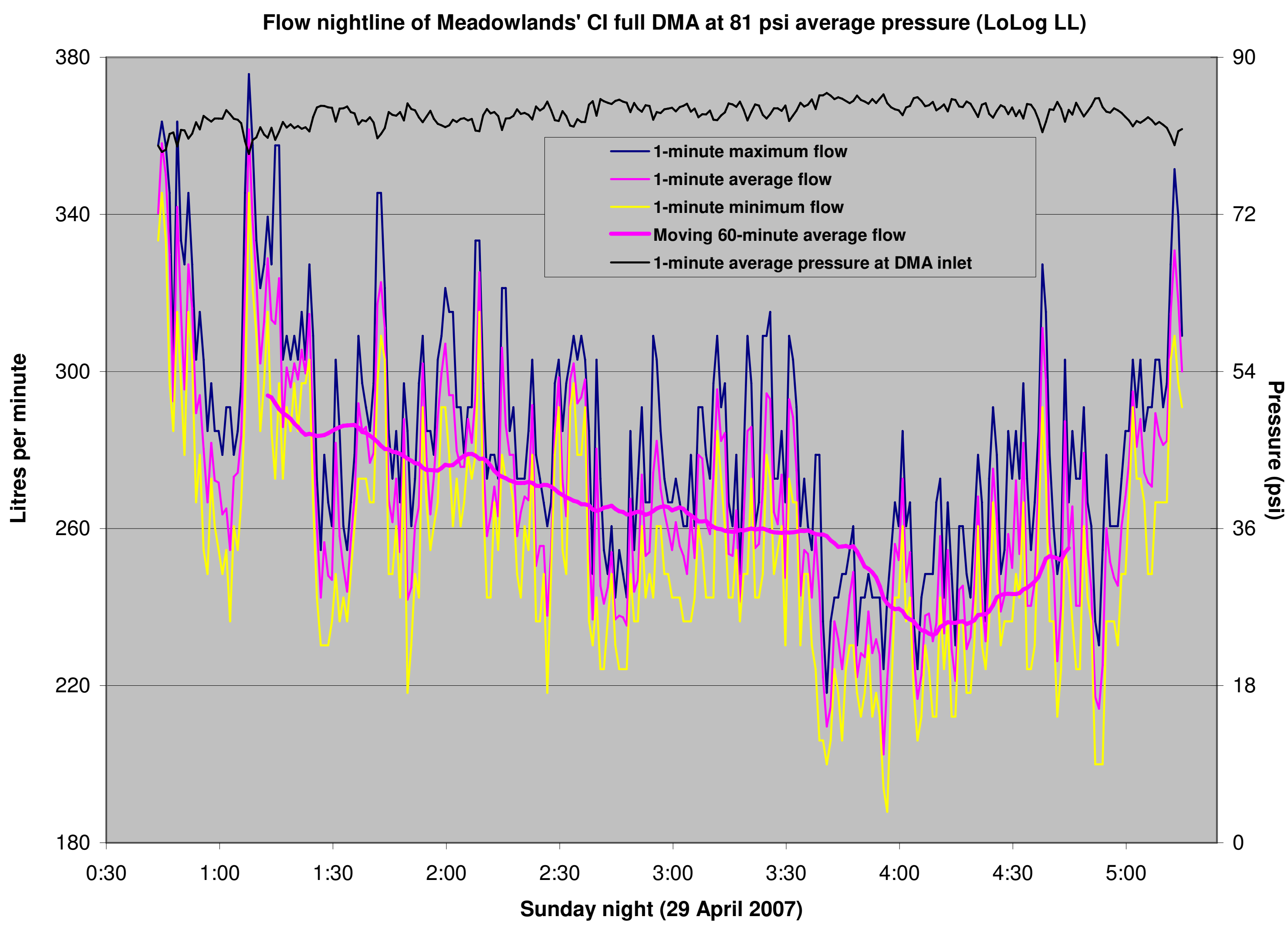

Flow nightline of Meadowlands' CI purple+blue sub-DMA at 63.5/76 psi average pressure (LoLog LL)
1-minute maximum flow
1-minute average flow
1-minute minimum flow
Moving 60-minute average flow
1-minute average pressure at DMA inlet
Litres per minute
Pressure (psi)
Wednesday night (25 April 2007)
0
20
40
60
80
100
120
0
15
30
45
60
75
90
0:00
0:30
1:00
1:30
2:00
2:30
3:00
3:30
4:00
4:30
5:00

Flow nightline of Meadowlands' CI purple+blue sub-DMA at 63.5/76 psi average pressure (LoLog LL)

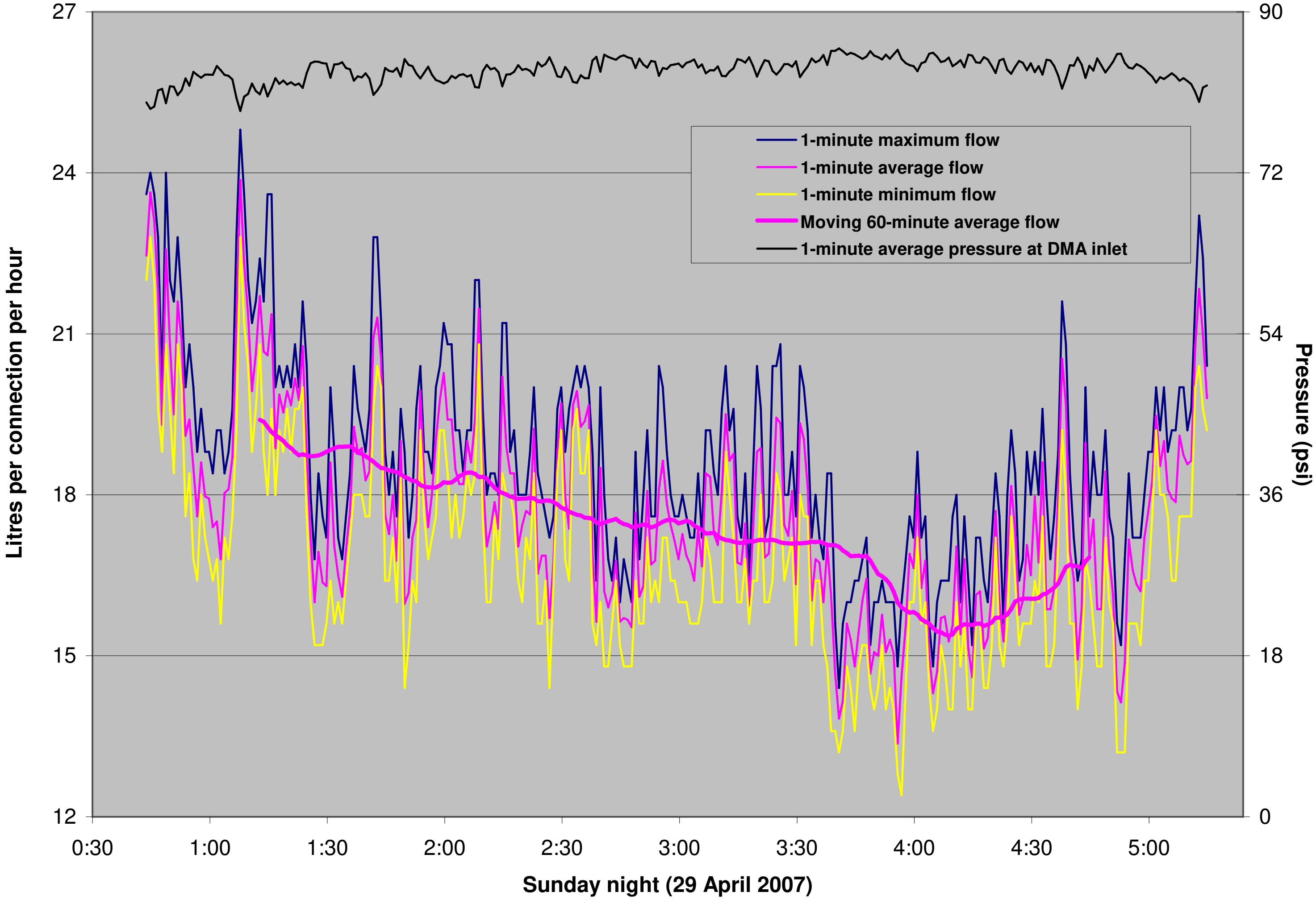


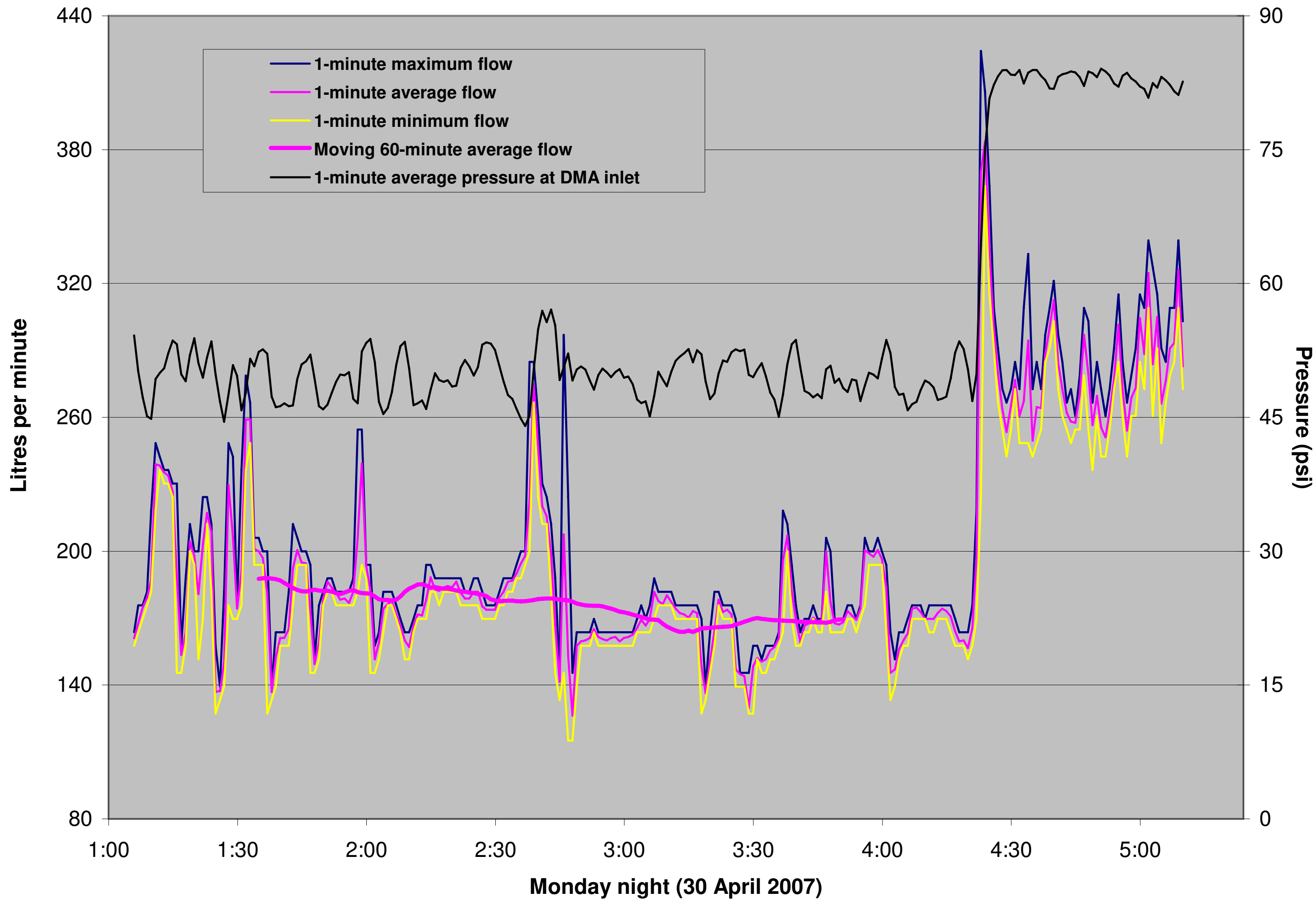

Flow nightline of Meadowlands' CI purple+blue sub-DMA at 48/63/81 psi average pressure (LoLog LL)
1-minute maximum flow
1-minute average flow
1-minute minimum flow
Moving 60-minute average flow
1-minute average pressure at DMA inlet
Litres per minute
Pressure (psi)
Thursday night (26 April 2007)
0
20
40
60
80
100
0
18
36
54
72
90
0:00
0:30
1:00
1:30
2:00
2:30
3:00
3:30
4:00
4:30
5:00


Flow nightline of Meadowlands' CI purple+blue sub-DMA at 48/63/81 psi average pressure (LoLog LL)
1-minute maximum flow
1-minute average flow
1-minute minimum flow
Moving 60-minute average flow
1-minute average pressure at DMA inlet
Litres per connection per hour
Pressure (psi)
Thursday night (26 April 2007)
0
4
8
12
16
20
0
18
36
54
72
90
0:00
0:30
1:00
1:30
2:00
2:30
3:00
3:30
4:00
4:30


Flow nightline of Meadowlands' CI full DMA at 81 psi average pressure (LoLog LL)
1-minute maximum flow
1-minute average flow
1-minute minimum flow
Moving 60-minute average flow
1-minute average pressure at DMA inlet
Litres per minute
Pressure (psi)
380
340
300
260
220
180
90
72
54
36
18
0
0:30
1:00
1:30
2:00
2:30
3:00
3:30
4:00
4:30
5:00
Sunday night (29 April 2007)

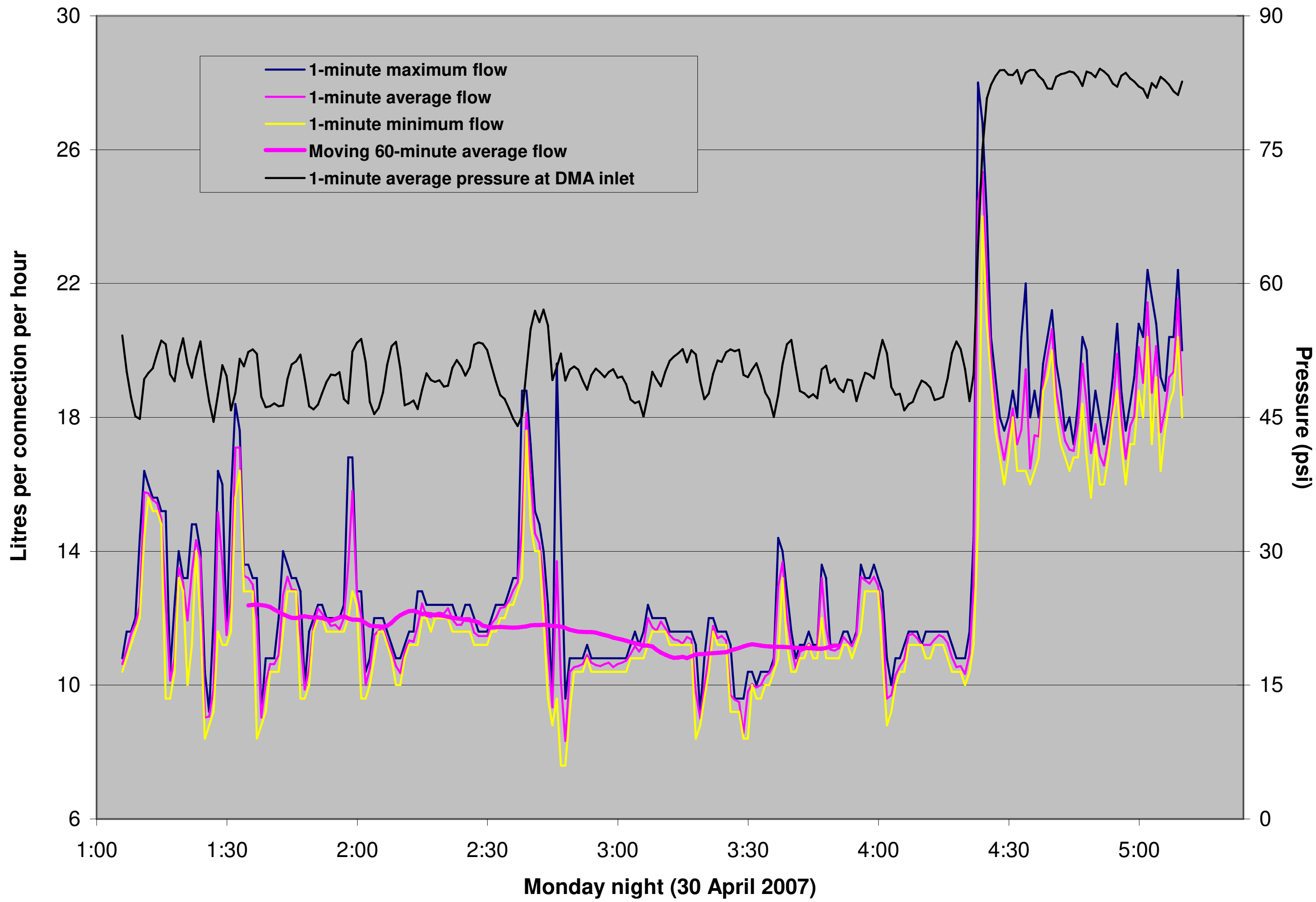
Flow nightline of Meadowlands' CI full DMA at 81 psi average pressure (LoLog LL)

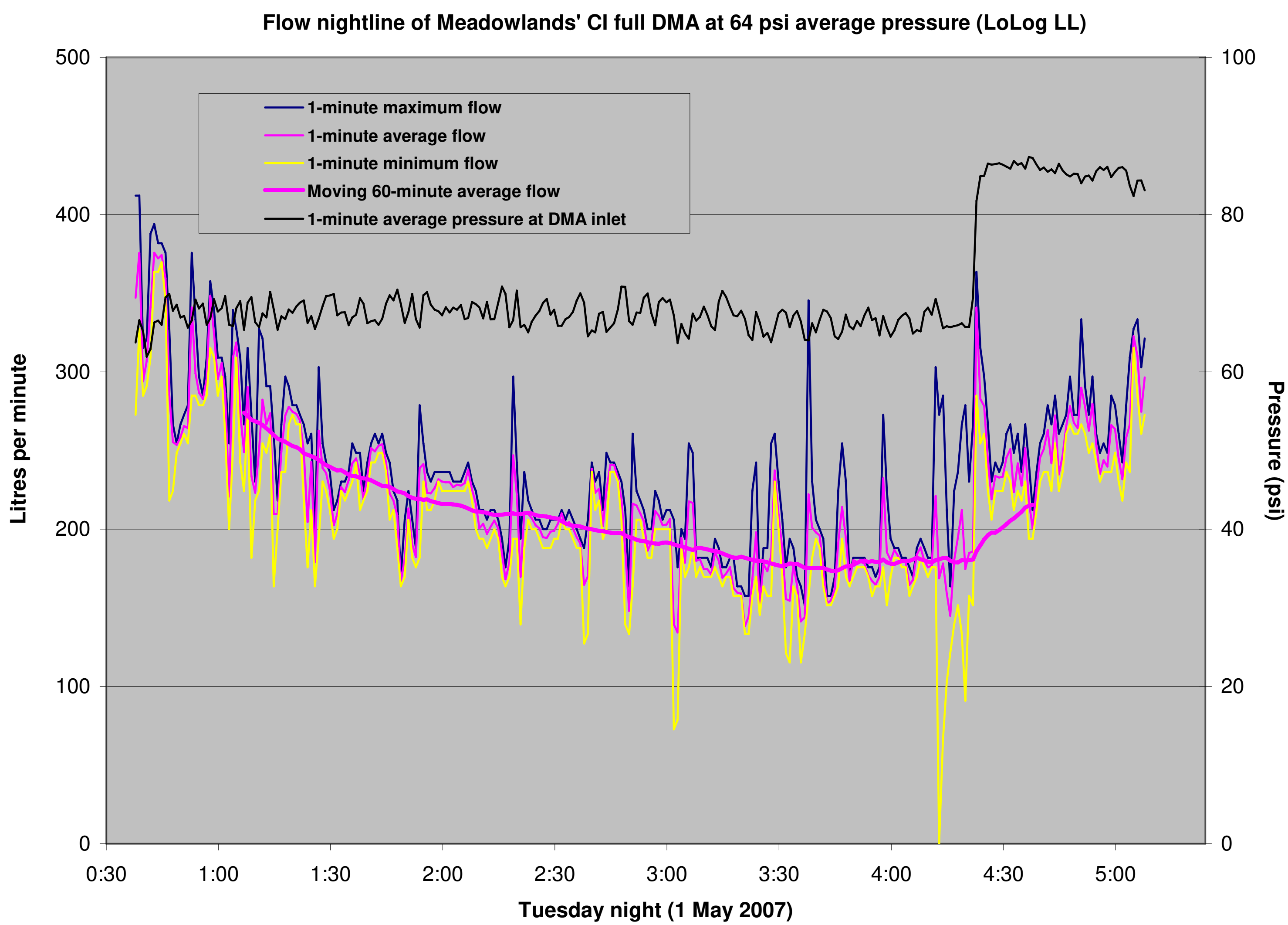




Flow nightline of Meadowlands' CI full DMA at 47 psi average pressure (LoLog LL)

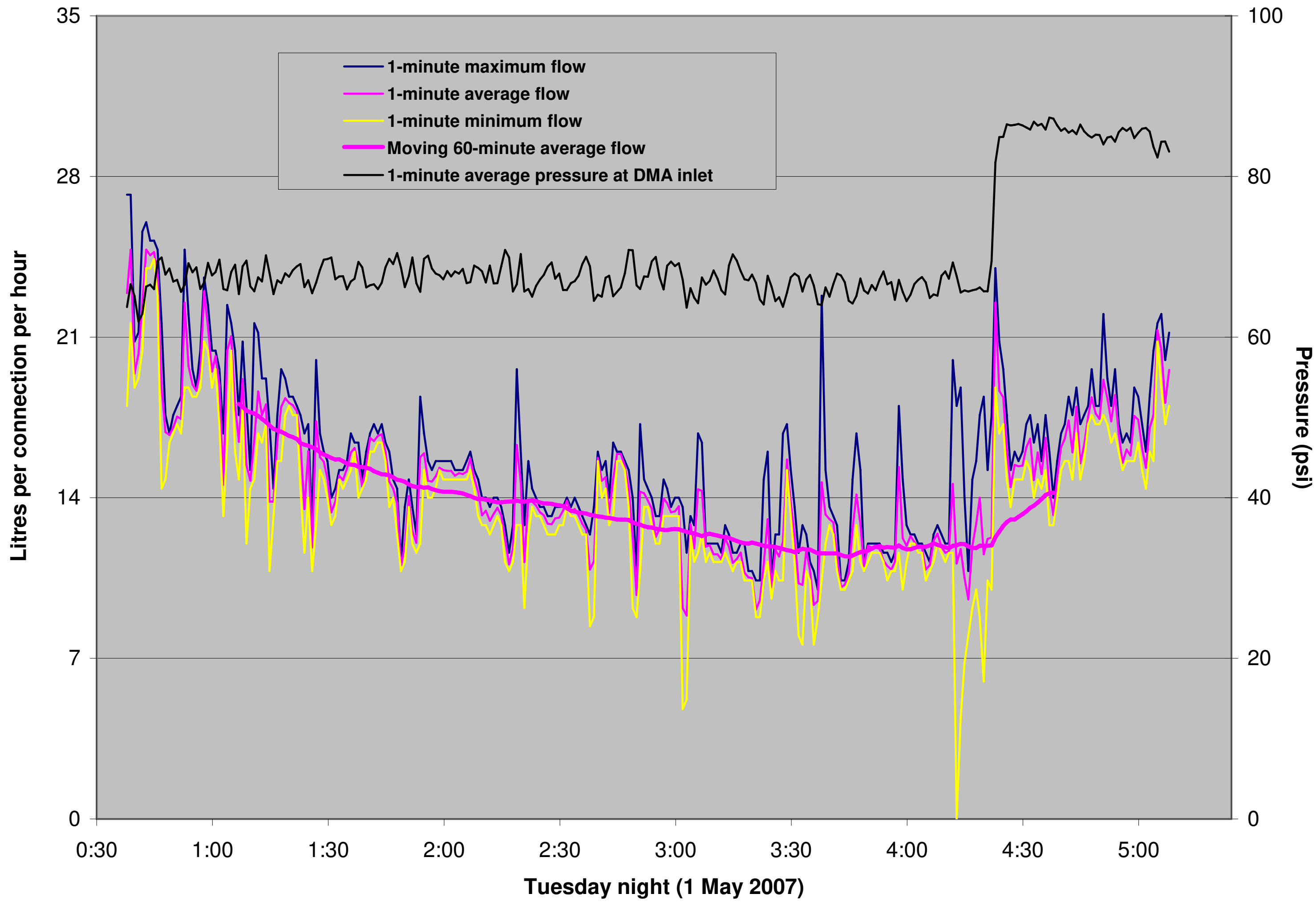
Flow nightline of Meadowlands' CI full DMA at 47 psi average pressure (LoLog LL)





Flow nightline of Meadowlands' CI full DMA at 64 psi average pressure (LoLog LL)
1-minute maximum flow
1-minute average flow
1-minute minimum flow
Moving 60-minute average flow
1-minute average pressure at DMA inlet
Litres per minute
Pressure (psi)
0
100
200
300
400
500
0
20
40
60
80
100
0:30
1:00
1:30
2:00
2:30
3:00
3:30
4:00
4:30
5:00
Tuesday night (1 May 2007)

Flow nightline of Meadowlands' CI full DMA at 64 psi average pressure (LoLog LL)



APPENDIX F

THEORETICAL ECONOMIC MODELS OF LEAK DETECTION METHODS FOR WATER DISTRIBUTION SYSTEMS

INTRODUCTION

Generally, it's claimed that leakage management strategies based on district-metered areas are economically more effective than strategies based on periodic acoustic surveys of the whole distribution system. This claim has not been validated by an economic analysis. It's probably based on the fact that district metering identifies high-leakage areas and subsequently the perception that it saves manpower that might be wasted on surveying low-leakage areas. This perception can be misleading since leaks in low-leakage areas will run for a longer time than those in high-leakage areas and subsequently may lead to a greater financial loss. Also, the initial capital cost of setting up DMAs and their maintenance is high, especially if distribution systems are to be retrofitted. This may offset the financial savings made by reducing the volume of lost water. Therefore, before a utility embarks on a DMA-based leakage management strategy, it's recommended that an analysis be undertaken to determine if DMAs are more economic than periodic acoustic surveys.

In this appendix, theoretical models are derived for calculating the total cost for leakage management strategies based on DMAs and periodic acoustic surveys. Minimum annual costs of these strategies and their corresponding water loss, i.e., economic leakage rates, are also derived. A parametric cost comparison of the two strategies is presented for Ottawa's water distribution system. This is followed by a discussion of non-economic factors that may affect the decision for selecting a leakage management strategy.

LEAKAGE RATE UNDER PERIODIC ACOUSTIC SURVEYS

Under periodic acoustic surveys, the total water loss from leaks in a distribution system during a complete survey cycle can be considered to be the summation of the following three components:

WL_{LBSF}: losses from leaks that develop behind the survey front

WL_{LASF}: losses from leaks that develop ahead of the survey front

WL_{LRPS}: losses from leaks remaining from the previous survey cycle

WL_{LBSF} is calculated as follows:

[1] $$WL_{LBSF} = \int_0^T \frac{L}{T}\, t\, F(t)\, (T-t)\, R\, dt$$

In words, this is the time integral of the number of leaks behind the survey front at time *t,* equal to the survey rate, $\frac{L}{T}$, where *L* is the system's total pipe length, in km, and *T* is the time taken to survey the whole network, i.e., survey period, in years, multiplied by *t* and the leak frequency, *F*(*t*), in leaks per km pipe per year; multiplied by the average duration of these leaks in the current survey cycle, equal to *T* – *t*; multiplied by the average leak flow rate, *R,* in m^3 per year per leak. Eq. [1] does not include water losses from background leaks, i.e., small leaks that are acoustically undetectable, and reported leaks, i.e., leaks that are visible to the naked eye or audible to the unaided human ear and are reported by the public or utility staff. WL_{LASF} and WL_{LRPS} are given by:

[2] $$WL_{LASF} = \int_0^T L(1-\frac{t}{T})\, F(t)\, \frac{(T-t)}{2}\, R\, dt$$

and

[3] $$WL_{LRPS} = \int_0^T \frac{L}{T}\, t\, F(t)\, \frac{t}{2}\, R\, dt$$

The leak frequency *F*(*t*) is assumed to vary linearly with time as follows:

[4] $$F(t) = F_o(1+\frac{mt}{100})$$

where *m* is the percent change of *F* per year, and F_o is the leak frequency at the start of the current survey cycle. The average flow rate of leaks, *R*, and the survey rate, $\frac{L}{T}$, are assumed to be constant over the duration of the survey. Losses due to the time taken to pinpoint and repair leaks are not included in Eqs. [1] to [3]; they are accounted for separately.

Substituting Eq. [4] in Eqs. [1] and [2] and $F(t) = F_o(1 - \frac{mT}{100} + \frac{mt}{100})$ in Eq. [3], and evaluating the integrals leads to:

[5] $$WL_{LBSF} = RLF_o T^2 (\frac{1}{6} + \frac{mT}{1200})$$

[6] $$WL_{LASF} = RLF_o T^2 (\frac{1}{6} + \frac{mT}{2400})$$

[7] $$WL_{LRPS} = RLF_o T^2 (\frac{1}{6} - \frac{mT}{2400})$$

The total water loss during a full survey cycle is the summation of loss components given by Eqs. [5] to [7]:

[8] $$WL_{total}^{cycle} = RLF_o T^2 (\frac{1}{2} + \frac{mT}{1200})$$

If the period of the leak detection cycle changes from T_1 to T_2, water loss in the first leak survey cycle at period T_2 due to leaks remaining from the last survey cycle at period T_1 is given by the following equation, instead of Eq. [3]:

[9] $$WL_{LRPS} = \int_0^{T_1} \frac{L}{T_1} t\, F(t) \frac{t}{2} \frac{T_2}{T_1} R\, dt = RLF_o T_1 T_2 (1 - \frac{mT_1}{2400})$$

and the total water loss from leaks in the first leak survey cycle at period T_2 is given by the following equation, instead of Eq. [8]:

[10] $$WL_{total}^{cycle} = RL\ F_o \left[T_2^2 (\frac{1}{3} + \frac{mT_2}{800}) + T_1 T_2 (\frac{1}{6} - \frac{mT_1}{1200}) \right]$$

If T_2 is shorter than T_1, e.g., as a result of the addition of more survey staff or improved efficiency, the total water loss in the first shorter survey period will be larger than losses in the following periods due to the larger number of leaks remaining from the last longer period.

The number of leaks that will be found during a complete survey cycle, N_{LFCC}, is the sum of the number of leaks that occur ahead of the survey front and the number of leaks that remain from the previous cycle and is given by:

$$N_{LFCC} = \int_0^T L(1-\frac{t}{T})F(t)dt + \int_0^T \frac{L}{T} tF(t)dt = \int_0^T LF(t)dt \quad [11]$$

Substituting Eq. [4] in Eq. [11] and evaluating the integral yield:

$$N_{LFCC} = LF_o T(1+\frac{mT}{200}) \quad [12]$$

Therefore, the number of leaks found per year is equal to LF_o (assuming $m = 0$), which is equal to the number of leaks that occur each year.

Alternatively, the total water loss in a complete leak survey cycle, WL_{total}^{cycle}, can be calculated as the integral of the total flow rate from all leaks at time t, R_{total}, in m^3 per year:

$$WL_{total}^{cycle} = \int_0^T R_{total}(t)\, dt \quad [13]$$

R_{total} is calculated as the number of leaks at time t, $N(t)$, multiplied by the average leak flow rate, R, in m^3 per leak per year, i.e.:

$$R_{total}(t) = N(t)R \quad [14]$$

The number of leaks at time t is given by:

$$N(t) = N_1 + N_2 - N_3 - N_4 \quad [15]$$

where :

N_1 is number of leaks remaining from the survey cycle that precedes the current one

N_2 is number of leaks that occur in a complete survey cycle

N_3 is number of leaks that remain from the survey cycle that precedes the current one and are found by time t in the current cycle

N_4 is number of leaks that occur in the current survey cycle and are found by time t

The above leak numbers are found using the following equations:

$$N_1 = \int_0^T \frac{L}{T} t\, F(t)\, dt \quad [16]$$

[17] $$N_2 = \int_0^T L\, F(t)\, dt$$

[18] $$N_3 = \int_0^t \frac{L}{T}\, \tau\, F(\tau)\, d\tau + \int_t^T \frac{L}{T}\, t\, F(\tau)\, d\tau$$

and

[19] $$N_4 = \int_0^t \frac{L}{T}\, (t-\tau)\, F(\tau)\, d\tau$$

The first part of the integral in Eq. [18] corresponds to the number of leaks remaining from the survey cycle that precedes the current one in pipe length $\frac{L}{T}t$ and which occur from time 0 to *t*; and the second part corresponds to the number of leaks remaining from the survey cycle that precedes the current one in pipe length $\frac{L}{T}t$ but which occur from time *t* to *T*. It is assumed that the survey cycles follow the same path.

Substituting Eq. [4] in Eqs. [17] and [19] and $F(t) = F_o(1 - \frac{mT}{100} + \frac{mt}{100})$ in Eqs. [16] and [18], and evaluating the integrals yields:

[20] $$N_1 = LF_oT(\frac{1}{2} - \frac{mT}{600})$$

[21] $$N_2 = LF_oT(1 + \frac{mt}{200})\frac{t}{T}$$

[22] $$N_3 = LF_oT(\frac{1}{2} - \frac{mT}{200} + \frac{mt}{300})\frac{t^2}{T^2} + LF_oT\left[(1 - \frac{mT}{200})\frac{t}{T} - (1 - \frac{mT}{100} + \frac{mt}{200})\frac{t^2}{T^2}\right]$$

and

[23] $$N_4 = LF_oT(\frac{1}{2} + \frac{mt}{600})\frac{t^2}{T^2}$$

Substituting Eqs. [20] to [23] in Eq. [15], Eq. [15] in Eq. [14], and Eq. [14] in Eq. [13] and evaluating the integral yield the following for total water loss in a full survey cycle:

[24] $$WL_{total}^{cycle} = RLF_oT^2(\frac{1}{2}+\frac{mT}{1200})$$

which is in agreement with Eq. [8].

For a constant leak frequency, i.e., $m = 0$, the total flow rate from unreported leaks at time t is found to be:

[25] $$R_{total}(t) = \frac{1}{2}RLF_oT$$

which is equal to the average yearly water loss from unreported leaks based on Eq. [24], i.e.:

[26] $$WL_{total}^{annual} = \frac{1}{2}R\ LF_oT$$

The leak frequency can be estimated from Eq. [12] since L and T are constant and N_{LFCC} is known from previous surveys. If m is equal to 0, the leak frequency can be found from the data of one survey cycle. If not, data from two cycles can be used to solve for m and F_o, assuming that m remains constant over the two cycles (and the intervening cycles). Assuming that $m = 0$, the leak frequency is given by:

[27] $$F_o = \frac{N_{LFCC}}{LT}$$

Substituting Eq. [27] in Eq. [8], the total water loss per complete survey cycle is equal to:

[28] $$WL_{total}^{cycle} = RN_{LFCC}\frac{T}{2}$$

OPTIMUM PERIOD FOR ACOUSTIC SURVEYS

The optimum period, $T^{optimum}$, for an acoustic survey is the period that minimizes the total yearly cost of the leakage management program. The total yearly cost consists of the cost of lost water and the cost of conducting the acoustic survey and, assuming a constant leak frequency, is given by:

[29]
$$\begin{aligned} C_{total}^{annual} &= \left[cRL\ F_o T^2 / 2 + C_{survey}\right] / T \\ &= cRL\ F_o T\ \ / 2 + C_{survey} / T \end{aligned}$$

where c is the marginal cost of lost water ($/m^3), R is the average volume of lost water in m^3 per year per leak (weighted according to frequencies of unreported distribution pipe leaks and service pipe leaks), L is the total length of distribution pipes in the whole system in km, F_o is the total frequency of unreported distribution and service pipes leaks per km of pipe per year, T is the survey period in years, and C_{survey} is the cost of acoustically surveying the whole distribution system.

The optimum period is found by setting the derivative of C_{total}^{annual}, with respect to T, to zero, i.e.:

[30] $$cRL\ F_o / 2 - C_{survey} / T^2 = 0$$

Hence:

[31] $$T^{optimum} = \sqrt{\frac{2C_{survey}}{cRLF_o}}$$

The cost of surveying the whole network, C_{survey}, is given by:

[32] C_{survey} = *Length of distribution pipes in km / (No. of survey teams* x *survey rate per team) x No. of teams x No. of persons per team x annual salary per person x overhead factor*

= *System length in km / survey rate (km /year / team) x No. of persons per team x annual salary per person x overhead factor*

The minimum annual cost of lost water and leak detection effort is obtained by substituting Eq. [31] in Eq. [29] as follows:

[33] $$C_{\min}^{annual} = \sqrt{2cRLF_o C_{survey}}$$

If the current annual leakage rate, $WL_{current}^{annual}$, and the current cost and period of surveying the whole network, C_{survey} and $T_{current}$, respectively, are known, then the optimum survey period given by Eq. [31] can also be expressed as follows:

[34] $$T^{optimum} = \sqrt{\frac{2C_{survey}T_{current}}{cRLF_oT_{current}}} = \sqrt{\frac{C_{survey}T_{current}}{cWL_{current}^{annual}}}$$

The current annual loss, $WL_{current}^{annual}$ in Eq. [34] does not include losses from background and reported leaks, nor does it include losses due to delays in pinpointing and repairing unreported leaks. Information about the frequency and size of leaks, which might be hard to obtain, is not needed for determining the optimum survey period using Eq. [34]. Similarly, the minimum annual cost of lost water and leak detection effort can be expressed as follows:

[35] $$C_{\min}^{annual} = \sqrt{2cRLF_o \frac{T_{current}}{T_{current}} \frac{2}{2} C_{survey}} = \sqrt{4cWL_{current}^{annual} \frac{C_{survey}}{T_{current}}}$$

OPTIMUM TIMING OF ACOUSTIC SURVEYS FOR DMAs

To determine the optimum timing of acoustic surveys for district metered areas, it's assumed that interventions to survey a DMA acoustically occur periodically, at the end of periods that are T_I long. The amount of water lost during the period T_I, assuming a constant leak frequency, is then given by:

[36] $$\begin{aligned} WL^{T_I} &= \int_0^{T_I} RL_{DMA}F_o(T_I - t)dt \\ &= RL_{DMA}F_oT_I^2/2 \end{aligned}$$

where R is the average volume of water lost in m^3 per year per leak, L_{DMA} is the length of distribution pipes in the DMA in km, F_o is the frequency of unreported leaks per km of pipe per year, and t is time.

The total yearly cost, C_{DMA}^{annual}, i.e., combined cost of lost water and acoustic surveys, is given by:

$$C_{DMA}^{annual} = \left[cRL_{DMA}F_oT_I^2 / 2 + C_{survey}^{DMA}\right] / T_I \qquad [37]$$
$$= cRL_{DMA}F_oT_I \; / 2 + C_{survey}^{DMA} / T_I$$

where c is the marginal cost of lost water (\$/m^3), and C_{survey}^{DMA} is the cost of acoustically surveying the whole DMA. The survey is assumed to be performed in a very short time compared to the period T_I, and hence the volume of water lost during the survey is assumed to be small and not taken into account.

The optimum intervention period, $T_I^{optimum}$, is the period that minimizes the total yearly cost. It's found by equating the derivative of the total yearly cost, C_{DMA}^{annual}, with respect to T_I, to zero, i.e.:

$$cRL_{DMA}F_o / 2 - C_{survey}^{DMA} / T_I^2 = 0 \qquad [38]$$

Hence:

$$T_I^{optimum} = \sqrt{\frac{2C_{survey}^{DMA}}{cRL_{DMA}F_o}} \qquad [39]$$

Eqs. [36] and [38] are the same as the equations that would have been obtained if acoustic surveys were performed uniformly over the period T_I. This may be taken as an indication that the uniformity of the survey is not necessary for these equations to hold. The cost of surveying the DMA is given by:

[40] C_{survey}^{DMA} = *Length of DMA's distribution pipes in km / survey rate (km / year /team)* x *No. of persons per team x annual salary per person x overhead factor*

The minimum annual combined cost of lost water and acoustic surveys is obtained by substituting Eq. [39] in Eq. [37] as follows:

$$C_{\min}^{annual} = \sqrt{2cRL_{DMA}F_oC_{survey}^{DMA}} \qquad [41]$$

From Eq. [39], it is found that:

[42] $$cRL_{DMA}F_o(T_I^{optimum})^2/2 = C_{survey}^{DMA}$$

The term on the left hand side of Eq. [42] is the volume of water lost during the period $T_I^{optimum}$. In words, Eq. [42] means that the most economic time to undertake an acoustic leak detection survey for DMAs is when the accumulated cost of lost water is equal to the cost of the survey. In the long term, the average length of the intervention period will be equal to $T_I^{optimum}$ given by Eq. [38]. However, as demonstrated further on in the paper, this intervention criterion alone does not lead to the minimum cost under most conditions.

LEAKAGE RATE UNDER PASSIVE LEAKAGE MANAGEMENT

Under a passive leakage management policy, only leaks that surface and subsequently are reported by the public or spotted by utility staff are repaired. Assuming that the average time taken by a leak to surface is $T_{surface}$, the yearly water loss is found by using Eq. [25] for periodic surveys at a period T=2$T_{surface}$ and is given by:

[43] $$\begin{aligned} WL_{passive}^{annual} &= R_{weighted}LF_o^{total}T/2 \\ &= R_{weighted}LF_o^{total}(2T_{surface})/2 \\ &= R_{weighted}L\ F_o^{total}T_{surface} \end{aligned}$$

where

[44] $$F_o^{total} = F_o^{mains} + F_o^{services}$$

and

[45] $$R_{weighted} = \frac{R_{mains}F_o^{mains} + R_{services}F_o^{service}}{F_o^{total}}$$

R_{mains} and $R_{services}$ are the average leak flow rate (m^3/year) for leaks in distribution pipes and service pipe leaks, respectively; F_o^{mains} and $F_o^{service}$ are leak frequencies (leaks/km of distribution pipes/year) for distribution pipes and service pipe leaks, respectively; and *L* is the length of distribution pipes in the network (km). The yearly total cost of lost water is given by:

[46] $$C_{passive}^{annual} = cR_{weighted} L F_o^{total} T_{surface} + \textit{cost of leak pinpointing}$$

where *c* is the marginal cost of lost water ($/m^3).

INTERVENTION CRITERIA FOR DMAs AND CORRESPONDING MINIMUM COST

The following three intervention criteria to survey DMAs for leaks are considered. It's assumed that the exit level, i.e., the leakage rate of a DMA at which acoustic leak surveys are concluded, is equal to the background leakage rate. It's also assumed that night flows of DMAs are continuously monitored via telemetry.

Criterion 1

Intervention to survey the whole DMA is triggered by the detection of a major leak,e.g., distribution pipe break, in the DMA's minimum night flow record. All leaks found by the survey are repaired. In the long-term, this intervention criterion is equivalent to surveying the DMA at time periods equal to:

[47] $$T_1 = \frac{1}{F_o^{mains} L_{DMA}}$$

where F_o^{mains} is the frequency of leaks in distribution pipes and L_{DMA} is the total length of distribution pipes in the DMA. The minimum total yearly cost of leakage management, i.e., combined cost of lost water and leak detection surveys excluding the cost of initial DMA setup, maintenance and night flow monitoring, is equal to:

[48] $$\begin{aligned} C_{\min}^{annual} &= \textit{cost of water lost due to mains leaks} + \textit{cost of mains leak detection surveys} \\ &\quad + \textit{cost of water lost due to service pipe leaks} \\ &= N_{DMA} c R_{mains} L_{DMA} F_o^{mains} (T_{awareness} + T_{location} + T_{repair}) + N_{DMA} L_{DMA} F_o^{mains} C_{survey}^{DMA} \\ &\quad + N_{DMA} c R_{services} L_{DMA} F_o^{services} \left(\frac{1}{F_o^{mains} L_{DMA}} \right) / 2 \end{aligned}$$

and the corresponding annual total water loss, excluding loss from background and reported leaks, is equal to:

[49]
$$WL_{total}^{annual} = N_{DMA} R_{mains} L_{DMA} F_o^{mains} (T_{awareness} + T_{location} + T_{repair}) + N_{DMA} R_{services} L_{DMA} F_o^{services} \left(\frac{1}{F_o^{mains} L_{DMA}} \right) / 2$$

where N_{DMA} is the total number of DMAs in the distribution system, c is the marginal cost of lost water ($/m^3), R_{mains} is the average flow rate for a mains leak (m^3/year), L_{DMA} is the length of distribution pipes (mains) in the DMA, F_o^{mains} is the leak frequency for distribution pipes (leaks / km / year), $T_{awareness}$ is the time it takes to detect the leak in minimum night flow record, in years, $T_{location}$ is the average time it takes to locate a leak, equal to one-half the time it takes to survey the whole DMA, T_{repair} is the wait time for the leak to be repaired, C_{survey}^{DMA} is the cost of acoustically surveying the whole DMA, $R_{services}$ is the average flow rate for a service pipe leak (m^3/year), and $F_o^{services}$ is the frequency of service pipe leaks (leaks / km of distribution pipe / year).

This criterion may be not lead to minimum cost if the optimum intervention time based on the frequency and size of service pipe leaks is less than the intervention interval given by Eq. [47], i.e.:

[50]
$$\sqrt{\frac{2C_{survey}^{DMA}}{cR_{service} L_{DMA} F_o^{service}}} \leq \frac{1}{F_o^{mains} L_{DMA}}$$

Possibly, this can be avoided if a secondary trigger occurs when the accumulated cost of lost water,excluding losses from background and reported leaks, monitored via continuous night flow measurement exceeds the cost of surveying the DMA. Frequently, however, especially in the case of large and frequent distribution pipe leaks, Criterion 1 is more economic than Criterion 3 and sometimes Criterion 2 below, which incorporate this secondary trigger. This is because for Criterions 1 and 2, the survey is synchronized with the time at which a large distribution pipe leak occurs. Subsequently, this reduces the duration of large leaks to at most few days, instead of half the optimum survey interval had their occurrence been assumed random (as when surveying without the aid of DMAs). Criterion 1 can be more economic than Criterion 2 when the inequality sign in Eq. [50] changes direction.

C r i t e r i o n   2

Intervention to survey DMAs is triggered by the detection of a major leak in the DMA's minimum night flow record. The DMA is first step-tested to narrow down the area of the leak and then the suspected sub-area is surveyed acoustically to locate the leak. In the long-term, this is equivalent to step-testing / surveying the DMA at time intervals given by Eq. [47]. Also, an intervention to survey the whole DMA is triggered when the accumulated cost of lost water, excluding losses from background and reported leaks, is equal to the cost of surveying the DMA. In the long-term, considering only service pipe leaks, this is equivalent to surveying the DMA at time intervals equal to, i.e.:

[51] $$T_{service}^{optimum} = \sqrt{\frac{2C_{survey}^{DMA}}{cR_{service}L_{DMA}F_o^{service}}}$$

The minimum total yearly cost of leakage management, i.e., combined cost of lost water and step-testing / leak detection surveys excluding the cost of the initial cost of DMA setup, and maintenance and night flow monitoring costs, is equal to:

[52] $$\begin{aligned} C_{\min}^{annual} &= \textit{cost of water lost due to mains leaks + Cost of mains step-tests / surveys} \\ &\quad + \textit{cost of water lost and leak detection surveys for service pipes} \\ &= N_{DMA}cR_{mains}L_{DMA}F_o^{mains}(T_{awareness} + T_{location} + T_{repair}) + N_{DMA}L_{DMA}F_o^{mains}C_{step-test/survey}^{DMA} \\ &\quad + N_{DMA}\sqrt{2cR_{service}L_{DMA}F_o^{service}C_{survey}^{DMA}} \end{aligned}$$

and the corresponding annual total water loss, excluding loss from background and reported leaks, is equal to:

[53] $$\begin{aligned} WL_{total}^{annual} &= N_{DMA}R_{mains}L_{DMA}F_o^{mains}(T_{awareness} + T_{location} + T_{repair}) \\ &\quad + N_{DMA}\sqrt{\frac{R_{service}L_{DMA}F_o^{service}C_{survey}^{DMA}}{2c}} \end{aligned}$$

Assuming that each DMA can be subdivided into *k* sub-areas, and assuming that the cost of step testing is 1/*l* the cost of surveying the whole DMA, then the cost of step-testing and surveying the suspected area for a major leak detected in the DMA's minimum night flow record is equal to:

[54] $$C^{DMA}_{step-test/survey} = \frac{1}{k} C^{DMA}_{survey} + C^{DMA}_{step-test} = \frac{1}{k} C^{DMA}_{survey} + \frac{1}{l} C^{DMA}_{survey} = \frac{k+l}{k\,l} C^{DMA}_{survey}$$

In Eq. [54], it's assumed that the whole sub-area where the leak is suspected will be surveyed acoustically since it may not be possible to distinguish the major leak from smaller ones. All leaks found in the sub-area will be repaired. The cost of water saved by repairing service leaks found in the sub-area is assumed to be small and hence not taken into account in Eq. [52]. Like Criterion 1, especially in the case of frequent large leaks in distribution pipes, Criterion 2 is often more economical than Criterion 3 below.

Criterion 3

Intervention to survey the whole DMA is triggered only when the accumulated cost of lost water, including that due to unreported leaks in both distribution and service pipes but excluding losses from background and reported leaks, is equal to the cost of surveying the DMA. In the long-term, this is equivalent to surveying the DMA at the end of time intervals equal to:

[55] $$T_I = \sqrt{\frac{2C^{DMA}_{survey}}{cR_{weighted} L_{DMA} F_o^{total}}}$$

where

[56] $$F_o^{total} = F_o^{mains} + F_o^{service}$$

and

[57] $$R_{weighted} = \frac{R_{mains} F_o^{mains} + R_{service} F_o^{service}}{F_o^{total}}$$

The minimum total yearly cost of leakage management, i.e., combined cost of lost water and leak detection surveys but excluding the cost of the initial cost of DMA setup, and maintenance and night flow monitoring costs, is equal to:

[58] $$C^{annual}_{\min} = \textit{cost of water lost due to mains leaks} + \textit{cost of water lost due to service leaks} + \textit{cost of surveying the whole DMA}$$

$$= N_{DMA} c R_{mains} L_{DMA} F_o^{mains} T_I / 2 + N_{DMA} c R_{service} L_{DMA} F_o^{service} T_I / 2 + N_{DMA} C_{survey}^{DMA} / T_I$$

$$= N_{DMA} c \left(\frac{R_{mains} F_o^{mains} + R_{service} F_o^{service}}{F_o^{total}} \right) L_{DMA} F_o^{total} T_I / 2 + N_{DMA} C_{survey}^{DMA} / T_I$$

and the corresponding annual total water loss, excluding loss from background and reported leaks, is equal to:

[59] $$WL_{total}^{annual} = N_{DMA} \left(\frac{R_{mains} F_o^{mains} + R_{service} F_o^{service}}{F_o^{total}} \right) L_{DMA} F_o^{total} T_I / 2$$

INITIAL SETUP AND MAINTENANCE COST OF EQUIPMENT

The initial setup cost of DMAs and acoustic leak detection equipment is factored into the annual cost of leakage management strategies by spreading it over several years. If the initial cost of equipment, P, is spread over n years at the utility's discount rate, r, the yearly cost, a, is given by:

[60] $$a = P \frac{r(1+r)^n}{(1+r)^n - 1}$$

The yearly maintenance cost of equipment is assumed to be equal to fixed percentage of its yearly cost.

DISCUSSION

Economics is not the only factor that influences the selection of a leakage management strategy. Legal, environmental, social and political factors may force decisions that lead to uneconomic leakage management strategies. Leaks can cause damage to roads creating hazardous conditions and they may flood building basements causing damage to property and stored goods. Also, when pressure in pipes drops below zero, e.g., due to pressure transients, pollutants may enter into pipelines creating major health risks. These problems have costly legal consequences. Also, high demand for water leads to excessive abstraction by water utilities from lakes and rivers, which may cause critical reduction in their levels. The negative impact of low water levels on aquatic life tarnishes the public image of water utilities and attracts the wrath of environmental activists. Finally, frequent or prolonged rationing of household water use dictated by persistent water shortages irritates the public and eventually leads to political pressure or action.

There may also be operational factors that affect the adoption of a particular leakage management strategy. For example, leak detection surveys are highly repetitive and may be boring. Therefore, the efficiency of the surveys may drop due to fading staff motivation, especially after long periods of low success in finding leaks. DMAs help to avoid this situation by directing leak detection staff to high-leakage areas, which keeps them challenged and motivated. Minimum night flows of DMAs can be used to assess the effectiveness of leak detection staff and equipment. If a distribution system is almost fully comprised of DMAs, the DMAs can also help to establish the system's leakage rate more accurately than traditional annual audits. The leakage rate of the distribution system is obtained by integrating continuously monitored minimum night flow rates over the whole year for each DMA and then summing up the results of all DMAs, which is known as the "bottom-up" approach. Accurate leakage rates help to evaluate the effectiveness of the adopted leakage management strategy and adjust it if necessary. Also, pressure-reducing valves can be easily combined with DMA hardware as part of a pressure management strategy to reduce leakage from background and other long-running leaks.

On the opposite side, leakage rates based on minimum night flow rates of DMAs may be subject to errors due to out-of-calibration meters, local variation in allowances for residential and non-residential night water use, uncertainty in background leakage rate and night-to-day flow rate conversion factor. Errors in minimum night flow rates lead to suboptimal timing of interventions to survey DMAs for leaks. Also, these errors propagate into leakage rates calculated using the bottom up-approach. DMAs usually have a large number of dead pipe ends. Therefore, water quality problems will arise if a regular pipe-flushing program is not implemented. Flushing programs increase the cost of DMAs significantly because boundary valves need to be opened and closed and then the DMA re-proved.

There can be significant errors in leakage rates based on DMA night flow rates during summer as a result of watering of lawns and filling of swimming pools at night. In Canada and other northern countries, similar errors can also arise during winter due to the practice of leaving water taps running slightly to prevent services from freezing during very cold spells. Therefore, during these seasons, there may be false calls for intervention to undertake leak surveys of DMAs, which causes cost to increase and de-motivates leak detection staff. Finally, unlike periodic leak surveys, DMA-based leakage management strategies require significant capital investment. This can deprive the utility of the flexibility it needs in case major adjustments or a change of the adopted strategy becomes necessary in the future.

Example using Ottawa's Water Distribution System

Ottawa's distribution system is comprised of 2,391 km of distribution pipes, of which 39% is cast iron, 34% is ductile iron, and 26% is PVC. The system has 168,704 service connections, with an average pipe length of 15 m, and it services 765,000 people. The average pressure in the system is 47.6 m (70 psi). The average volume of water pumped into the system is 368 ML/day, and the average volume delivered is 312.6 ML/day. The current leakage management strategy is passive. The infrastructure is assumed to be in an average condition. The system's marginal cost of water is 4.6 cents/m^3.

Input Parameters

Each DMA is assumed to consist of 2000 service connections, and hence the total number of DMAs is 84. It's also assumed that each DMA can be divided into 5 sub-areas for step-testing and the cost of step-testing is equal to 1/5th the cost of acoustic surveys. Leak frequencies of distribution pipes are assumed to be 0.24, 0.064,and 0.006 leaks/km/year for cast iron, ductile iron, and PVC pipes, respectively. It's assumed that 50% of distribution pipe leaks are unreported and the average leak size is 65.7 ML/year (150 L/minute, based on a night-to-day flow rate conversion factor equal to 20). It's also assumed that the leak frequency of service connection pipes is 0.5 leaks/km/year (distribution pipe kms), 50% of leaks are unreported and the average leak size is 11.8 ML/year (27 L/minute). It's assumed that it takes 3 and 14 days to pinpoint and repair unreported leaks in distribution and service pipes, respectively; reported leaks are assumed to take 1 and 7 days for distribution and service pipes, respectively. The cost of leak repair is assumed to be independent of the leakage management strategy and hence not considered in the analysis. Under a passive leakage management strategy, leaks that don't surface instantly are assumed to surface in 2 years.

Equipment and maintenance cost for each DMA is $7222/year based on the following assumptions: initial setup cost is $75,000, service life is 20 years, maintenance cost is 20% of amortized initial cost, and discount rate is 5%. Cost for all DMAs is $606,648/year. Equipment and maintenance cost for a 2-person leak correlation team is $23,559/year based on the following assumptions: initial setup cost is $85,000 (2 vehicles at $30,0000 each and 1 correlator at $25,000), service life is 5 years, maintenance cost is 20% of amortized initial cost, and discount rate is 5%. Equipment and maintenance cost for a 1-person correlation team is $15,244/year based on the following assumptions: initial setup cost is $55,000 (1 vehicle at $30,0000 and 1 correlator at $25,000), service life is 5 years, maintenance cost is 20% of amortized initial cost, and discount rate is 5%. Equipment and maintenance cost for a 1-person leak sounding team is $9,700/year based on the following assumptions: initial setup cost is $35,000 (1 vehicle at $30,0000 and 1 listening device at $5,000), service life is 5 years, maintenance cost is 20% of amortized initial cost, and discount rate is 5%.

The cost of conducting correlation-based surveys is $107.5/km, or $128/km if equipment and maintenance cost is included, based on the following assumptions: time spent per correlation is 12 minutes, average distance between correlation points is 150 m, net time worked is 1488.5 hours/year (which leads to a survey rate of 1116 km/year/team), salary is $40,000/year/person, overhead cost is 50% of salaries, and each survey team consists of 2 persons. The labour cost of surveying the whole distribution system is $257,010 and the cost of surveying a DMA is $3,047, or $1,219 if a DMA step-test is performed first.

The cost of acoustic listening (sounding) surveys is $203/km, or $241/km if equipment and maintenance cost is included, based on the following assumptions: time spent listening per service is 5 minutes, net time worked is 1488.5 hours/year (leading to a survey rate of 253.2 km/year/team or 17862 services/year/team), salary is $40,000/year/person, overhead cost is 50% of salaries, and each survey team consists of 1 person. The labour cost of sounding the whole system is $485,154 and the cost of surveying a DMA is $5,752, or $1,725 if a DMA step-test is performed first.

The cost of pinpointing both unreported and reported leaks, excluding equipment cost, is $120,000/year based on the following assumptions: salary is $40,000/year/person, overhead cost is 50% of salaries, each survey team consists of 1 person and number of teams is 2.

Results

Minimum yearly total costs for periodic acoustic surveys and DMA-based leakage management strategies, as well as the corresponding costs of lost water and active leakage control effort (ALC), are listed in Table 1. It should be noted that the cost of losses from background and reported leaks are not included in total costs in order to emphasize the influence of unreported leaks. The following performance indicators are also listed in the Table 1: (i) volume of lost water from reported, unreported and background leaks, as percentage of total system input, and (ii) Infrastructure Leakage Index (ILI), calculated in accordance with IWA's performance indicators manual (Alegre et al., 2000). For acoustic surveys, the cost and performance indicators are also listed for a survey period that is twice the optimum one. For the sake of comparison, results for a marginal costs of water of ¢25/m^3 and ¢50/m^3 are listed in Tables 2 and 3, respectively. The number of needed survey teams for the case of ¢4.6/m^3 marginal cost of water is listed in Table 4.

For a marginal cost of water of ¢4.6/m^3, considered to be the Reference Case, a strategy based on periodic correlation surveys (at a 10 months period) is found to be the most economic option. However, for marginal costs of ¢25/m^3 and ¢50/m^3, the cost of a DMA-based strategy (with intervention criterion 1) is almost the same as the cost of periodic correlation surveys. As can be seen from Figure 1, periodic surveys remain the most economic option up to a marginal cost of water that is equal to approximately ¢75/m^3. At the economic leakage rate, i.e., that corresponding to minimum total cost, performance indicators of periodic surveys are higher than those of DMAs for marginal cost of water of ¢4.6/m^3, except for Criterion 3 which is equivalent to periodic surveys. As the marginal cost of water increases, the disparity between performance indicators of the two strategies becomes narrower. At the economic leakage rate, required human resources for leak detection are almost the same for periodic surveys and DMAs, except for Criterion 1 for which they are higher.

Periodic surveys become more economic than DMAs at high marginal costs of water if the leak frequencies for distribution pipes were lower than the values assumed above. This can be seen from Figure 2, which shows the minimum yearly total cost when a leak frequency of 0.1 leaks/km/year is used for cast iron pipes instead of the frequency of 0.24 assumed for the Reference Case in Figure 1. As noted earlier, DMAs with Criterion 1 and 2 work best in the case of frequent large leaks in distribution pipes. This is because the leak survey is synchronized with the time at which a distribution pipe leak occurs, i.e., the duration of large leaks is reduced to, at most, few days. This also explains why DMAs become more economic as the cost of marginal water increases. The advantage of synchronizing leak surveys with the occurrence of major leaks diminishes as the leak frequency for distribution pipes becomes smaller or as the majority of leaks are in service pipes. However, if the number of service pipe leaks decreases relative to distribution pipe leaks, periodic surveys become less economic than DMAs. This can be seen from Figure 3, which shows the minimum yearly total cost when a leak frequency of 0.25 leaks/km/year is used for service pipes instead of the value of the frequency of 0.5 used for the Reference Case in Figure 1.

Periodic surveys also become more economic than DMAs at high marginal costs of water as the salaries of leak surveys staff become lower. This can be seen from Figure 4, which shows the minimum total cost when a staff salary of $20,000/year is used instead of the salary of $40,000/year used for the Reference case. Figure 5 shows the minimum yearly total cost for a longer time to perform the correlation operation of 30 minutes instead of the 12 minutes used for the Reference Case. It can be seen that as a result of the increased duration, periodic surveys became less economic than DMAs at high marginal costs of water.

Figures 6 to 10 show minimum yearly total costs for periodic surveys and DMAs consisting of 1000, 3000, 4000, 5000 and 10,000 properties, respectively. In reference to the results of 2000-property (i.e., service connection) DMAs shown in Figure 1, periodic surveys are more economic than 1000-property DMAs at both low and high marginal costs of water, as can be seen in Figure 6. However, as the DMA size is increased from 2000 to 3000, 4000 or 5000 properties, the marginal cost of water beyond which periodic surveys is less economic than DMAs becomes lower, as can be seen in Figures 7 to 9. It’s interesting to note that as the number of properties in DMAs increases, the marginal cost of water beyond which a DMA-based strategy becomes more economic than periodic surveys also increases. As the size of DMAs increases to 10,000 properties, the minimum yearly total costs of leakage management strategies based on periodic surveys and DMAs (with Criterion 3) become nearly equal, as can be seen in Figure 10. Periodic surveys are more economic than very small DMAs because of the increased number of the latter and subsequently the higher total initial setup cost. The economic advantage of DMAs diminishes as their size increases because the cost of surveying DMAs for large leak events also increases with the size of the DMA. Intervention Criterion 1 or 2 are more economic than Criterion 3 over most marginal costs of water, except for very large DMAs or low leak frequencies for distribution pipes.

Important note: The above results are based on unconfirmed parameters and should not be used to select a leakage management strategy.

Table 1: Minimum Cost and Performance Indicators of Leakage Management Strategies for ¢4.6/m^3 Marginal Cost of water

(a) Correlation-based acoustic surveys are used for both DMAs and periodic surveys

	DMAs					
	Intervention criterion			Periodic acoustic surveys		
Cost ($/yr) / PI	**1**	**2**	**3**	**10 months (optimum)**	**20 months**	**Passive control**
Total	$1,389,746.12	$1,416,114.33	$1,456,371.20	$847,195.16	$971,745.96	$1,661,653.68
Lost water	$120,581.38	$224,821.87	$325,899.02	$325,899.02	$635,853.55	$1,511,165.01
ALC	$1,269,164.73	$1,191,292.45	$1,130,472.18	$521,296.14	$335,892.40	$150,488.67
% loss	7.0	8.7	10.3	10.3	14.8	25.5
ILI	2.0	2.6	3.1	3.1	4.7	9.2

(b) Acoustic listening (sounding) surveys are used for both DMAs and periodic surveys

	DMAs					
	Intervention criterion			Periodic acoustic surveys		
Cost ($/yr) / PI	**1**	**2**	**3**	**14 months (optimum)**	**28 months**	**Passive control**
Total	$1,842,474.15	$1,668,952.28	$1,707,745.63	$1,098,569.58	$1,271,284.80	$1,661,653.68
Lost water	$127,043.42	$303,844.37	$441,800.11	$441,800.11	$867,655.73	$1,511,165.01
ALC	$1,715,430.73	$1,365,107.91	$1,265,945.52	$656,769.47	$403,629.07	$150,488.67
% loss	7.1	9.9	12.0	12.0	17.9	25.5
ILI	2.1	3.0	3.7	3.7	5.9	7.8

Table 2: Minimum Cost and Performance Indicators of Leakage Management Strategies for ¢25/m^3 Marginal Cost of water

(a) Correlation-based acoustic surveys are used for both DMAs and periodic surveys

	DMAs					
	Intervention criterion			Periodic acoustic surveys		
Cost ($/yr) / PI	**1**	**2**	**3**	**4 months (optimum)**	**8 months**	**Passive control**
Total	$1,924,498.34	$2,104,297.38	$2,433,354.16	$1,824,178.11	$2,114,538.66	$8,363,341.99
Lost water	$655,333.61	$587,444.19	$809,240.08	$809,240.08	$1,531,825.31	$8,212,853.31
ALC	$1,269,164.73	$1,516,853.19	$1,624,114.07	$1,014,938.03	$582,713.35	$150,488.67
% loss	7.0	6.8	7.4	7.4	9.6	25.5
ILI	2.0	2.0	2.2	2.2	2.9	9.2

(b) Acoustic listening (sounding) surveys are used for both DMAs and periodic surveys

	DMAs					
	Intervention criterion			Periodic acoustic surveys		
Cost ($/yr) / PI	**1**	**2**	**3**	**6 months (optimum)**	**12 months**	**Passive control**
Total	$2,405,884.10	$2,589,299.61	$3,019,373.80	$2,410,197.76	$2,812,842.18	$8,363,341.99
Lost water	$690,453.37	$779,688.37	$1,079,435.88	$1,079,435.88	$2,072,216.91	$8,212,853.31
ALC	$1,715,430.73	$1,809,611.24	$1,939,937.92	$1,330,761.87	$740,625.27	$150,488.67
% loss	7.1	7.4	8.3	8.3	11.1	25.5
ILI	2.1	2.1	2.4	2.4	3.4	9.2

Table 3: Minimum Cost and Performance Indicators of Leakage Management Strategies ¢50/m^3 Marginal Cost of water

(a) Correlation-based acoustic surveys are used for both DMAs and periodic surveys

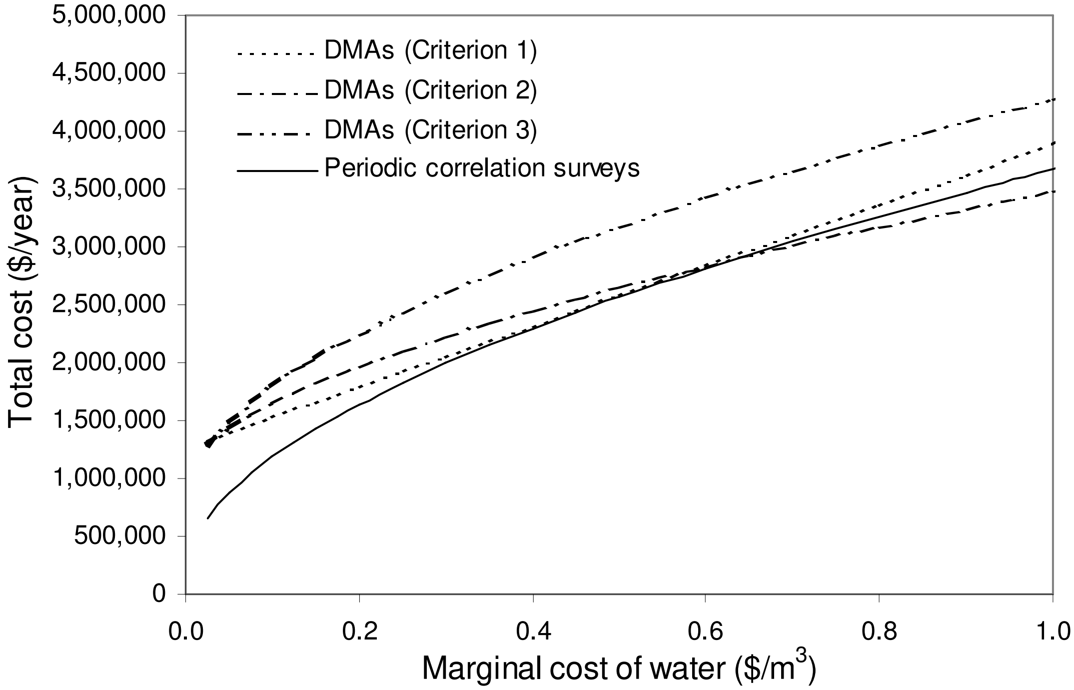
	DMAs					
	Intervention criterion			Periodic acoustic surveys		
Cost ($/yr) / PI	**1**	**2**	**3**	**3 months (optimum)**	**6 months**	**Passive control**
Total	$2,579,831.95	$2,648,732.44	$3,177,380.26	$2,568,204.22	$2,978,836.04	$16,576,195.30
Lost water	$1,310,667.22	$895,731.45	$1,195,199.55	$1,195,199.55	$2,217,089.37	$16,425,706.62
ALC	$1,269,164.73	$1,753,000.98	$1,982,180.72	$1,373,004.67	$761,746.67	$150,488.67
% loss	7.0	6.3	6.8	6.8	8.3	25.5
ILI	2.0	1.8	2.0	2.0	2.5	9.2

(b) Acoustic listening (sounding) surveys are used for both DMAs and periodic surveys

	DMAs					
	Intervention criterion			Periodic acoustic surveys		
Cost ($/yr) / PI	**1**	**2**	**3**	**4 months (optimum)**	**8 months**	**Passive control**
Total	$3,096,337.47	$3,307,869.73	$4,006,137.19	$3,396,961.15	$3,966,386.35	$16,576,195.30
Lost water	$1,380,906.74	$1,175,834.85	$1,577,314.11	$1,577,314.11	$2,981,318.50	$16,425,706.62
ALC	$1,715,430.73	$2,132,034.88	$2,428,823.08	$1,819,647.04	$985,067.86	$150,488.67
% loss	7.1	6.8	7.4	7.4	9.5	25.5
ILI	2.1	2.0	2.2	2.2	2.8	9.2

Table 4: Number of required survey teams for ¢4.6/m^3 marginal cost of water

	Correlation surveys (2-person teams)	Sounding surveys (1-person teams)
Periodic surveys	2.6	8.3
DMAs (Criterion 1)	3.5	15.7
DMAs (Criterion 2)	2.4	8.6
DMAs (Criterion 3)	2.6	8.3



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Total cost ($/year)
DMAs (Criterion 1)
DMAs (Criterion 2)
DMAs (Criterion 3)
Periodic correlation surveys
0.0
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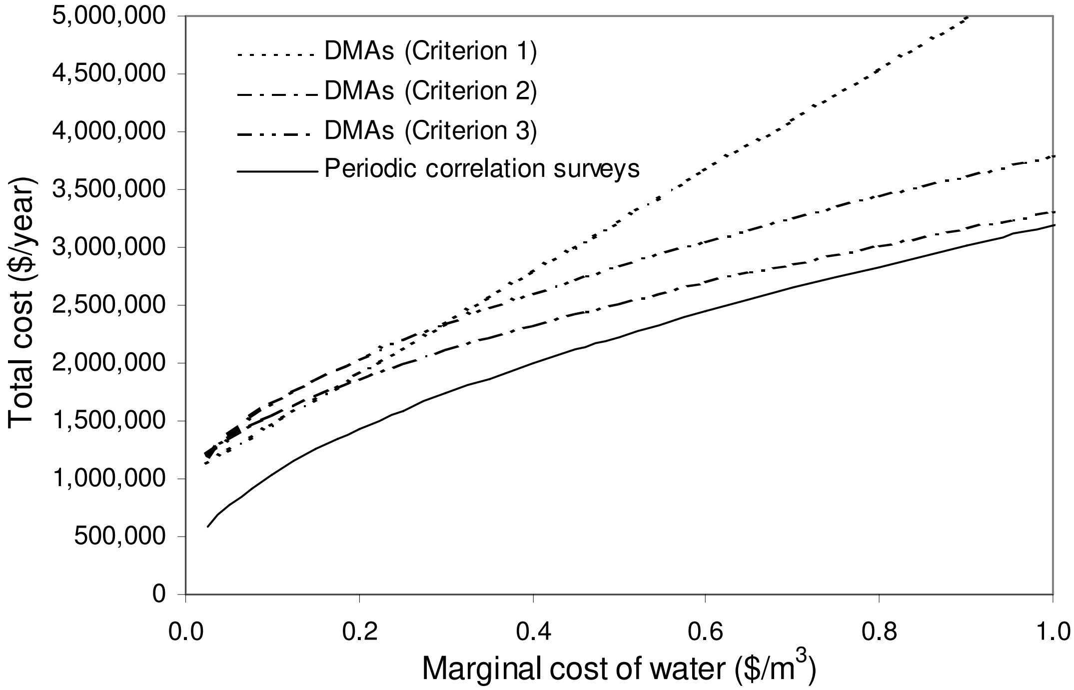
0.8
1.0
Marginal cost of water ($/m3)

Figure 1



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Total cost ($/year)
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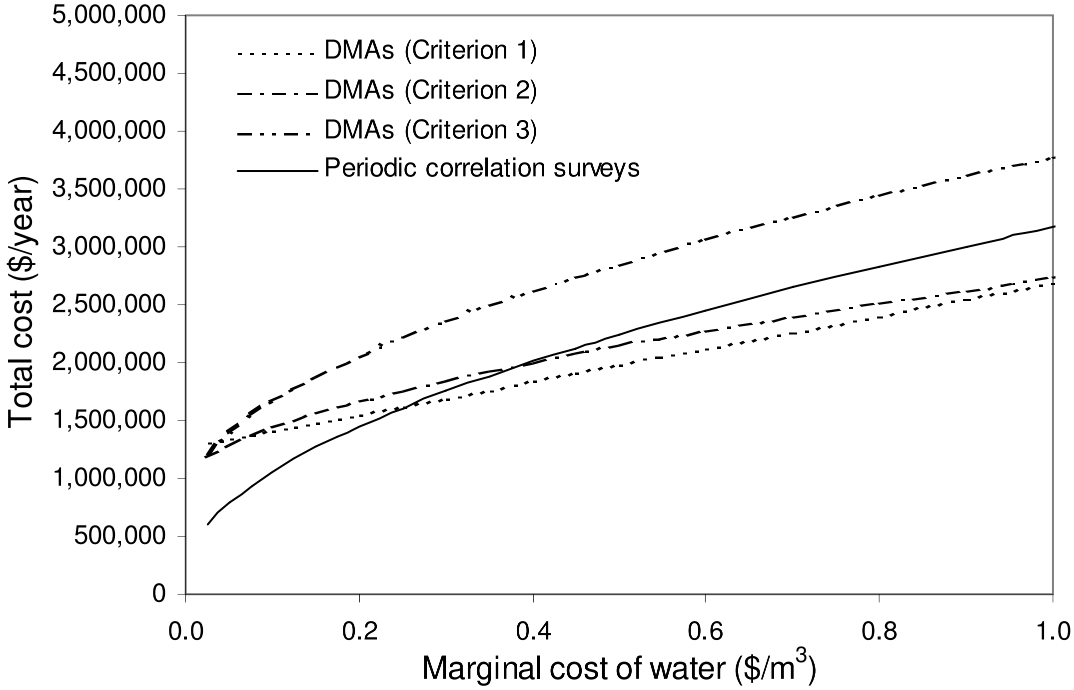
Marginal cost of water ($/m3)
DMAs (Criterion 1)
DMAs (Criterion 2)
DMAs (Criterion 3)
Periodic correlation surveys

Figure 2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Total cost ($/year)
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0.4
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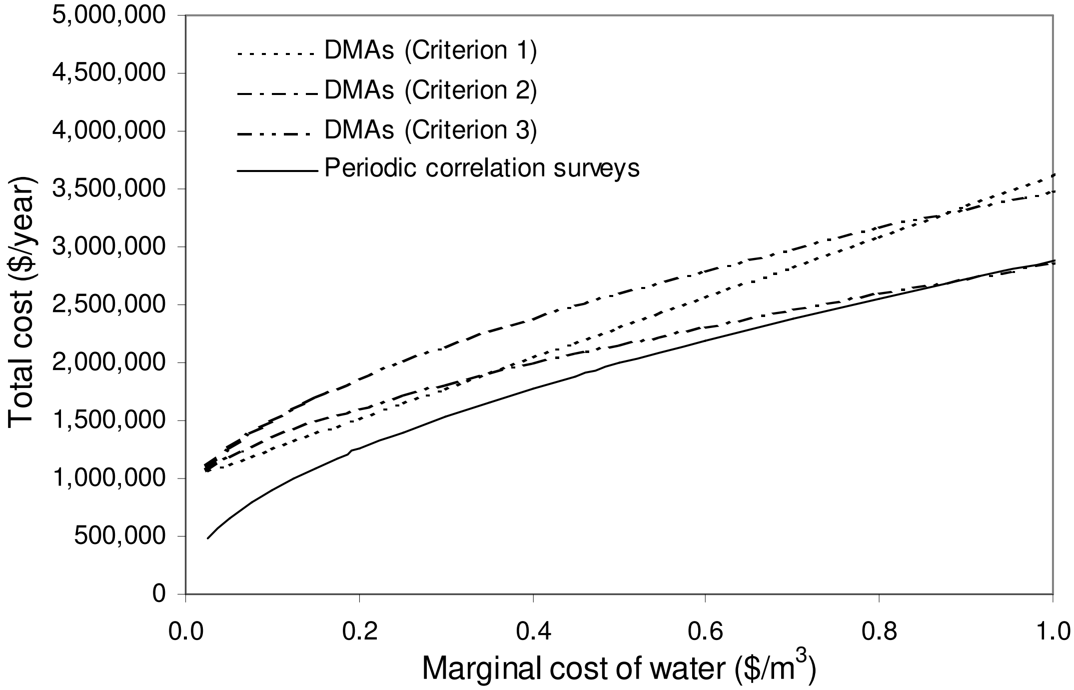
Marginal cost of water ($/m3)
DMAs (Criterion 1)
DMAs (Criterion 2)
DMAs (Criterion 3)
Periodic correlation surveys

Figure 3


DMAs (Criterion 1)
DMAs (Criterion 2)
DMAs (Criterion 3)
Periodic correlation surveys
Total cost ($/year)
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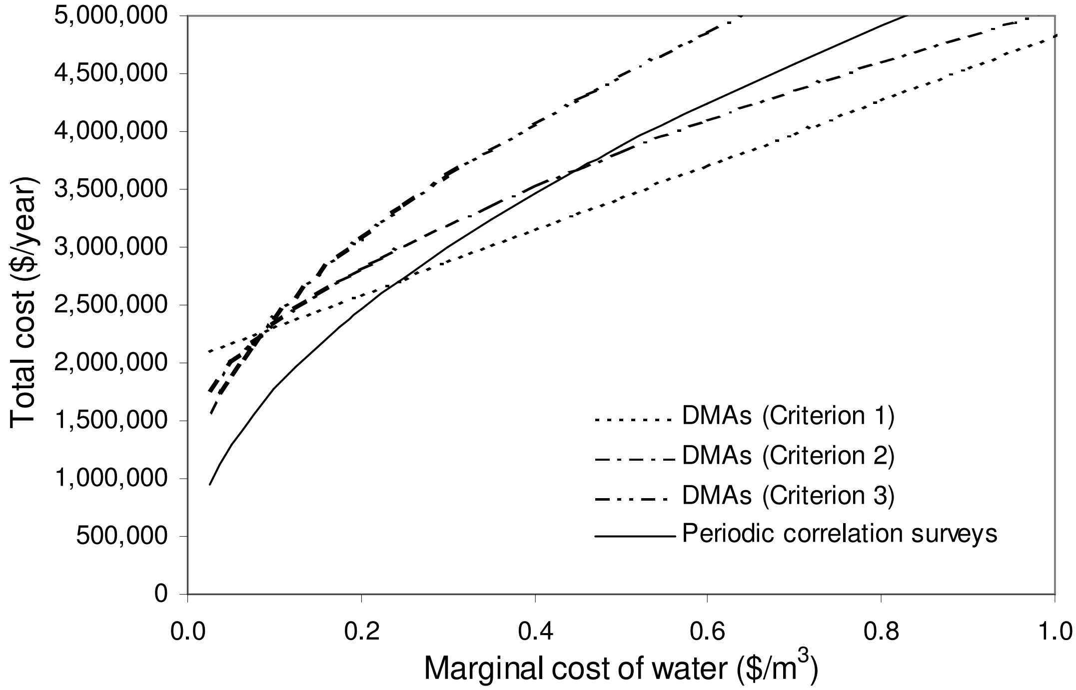
Marginal cost of water ($/m3)

Figure 4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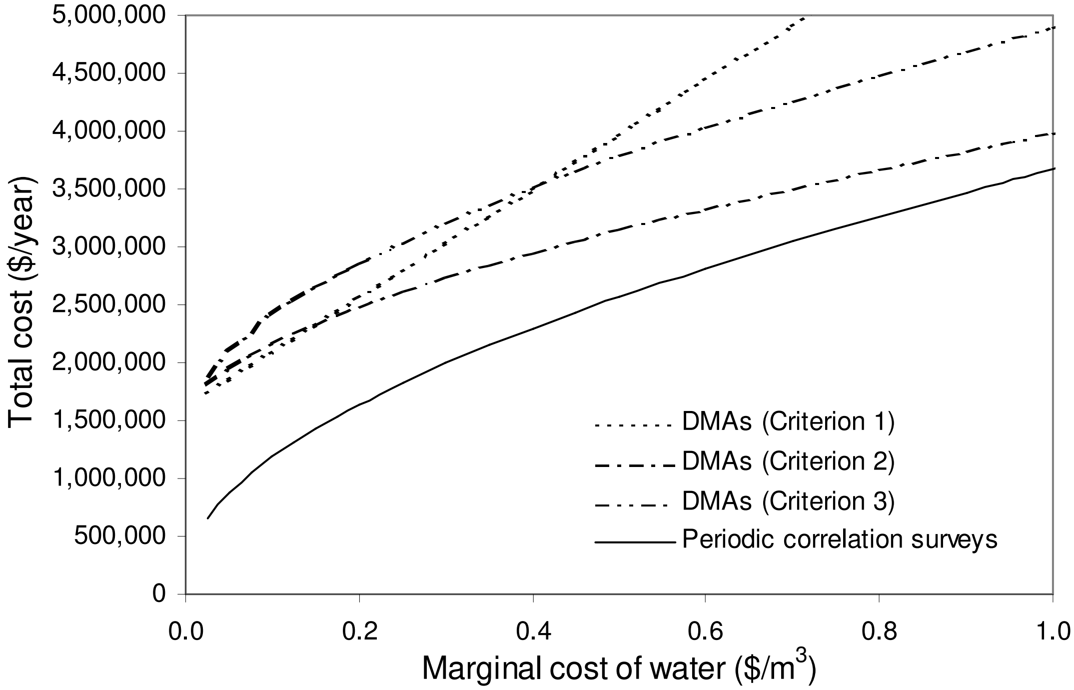
Total cost ($/year)
0.0
0.2
0.4
0.6
0.8
1.0
Marginal cost of water ($/m3)
DMAs (Criterion 1)
DMAs (Criterion 2)
DMAs (Criterion 3)
Periodic correlation surveys

Figure 5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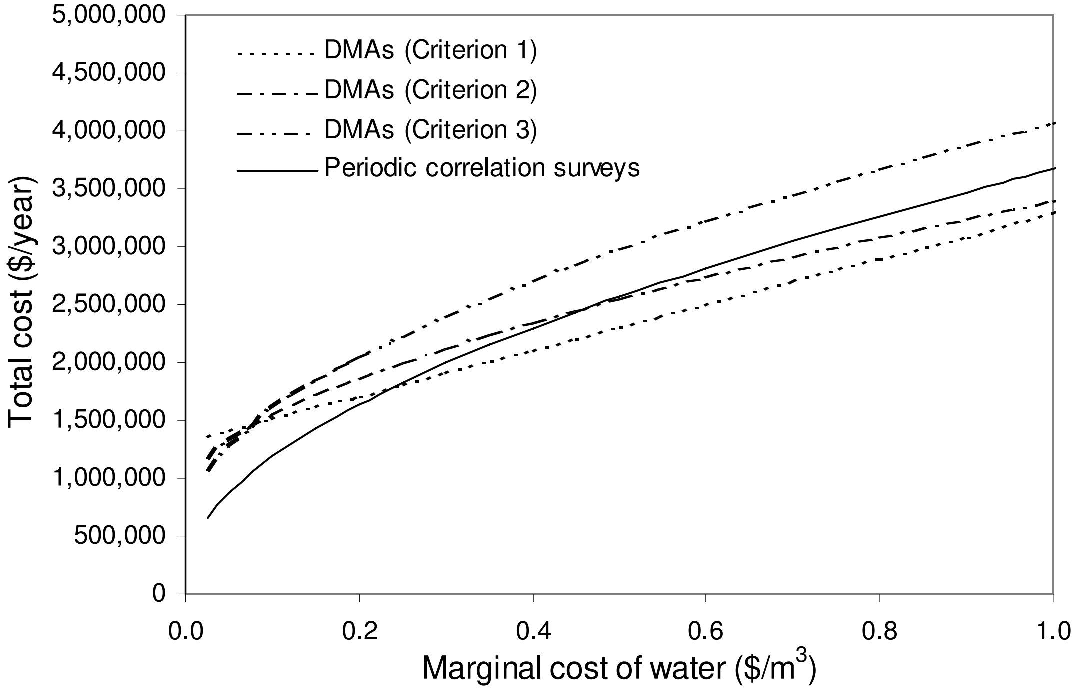
Total cost ($/year)
0.0
0.2
0.4
0.6
0.8
1.0
Marginal cost of water ($/m3)
DMAs (Criterion 1)
DMAs (Criterion 2)
DMAs (Criterion 3)
Periodic correlation surveys

Figure 6



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Total cost ($/year)
DMAs (Criterion 1)
DMAs (Criterion 2)
DMAs (Criterion 3)
Periodic correlation surveys
0.0
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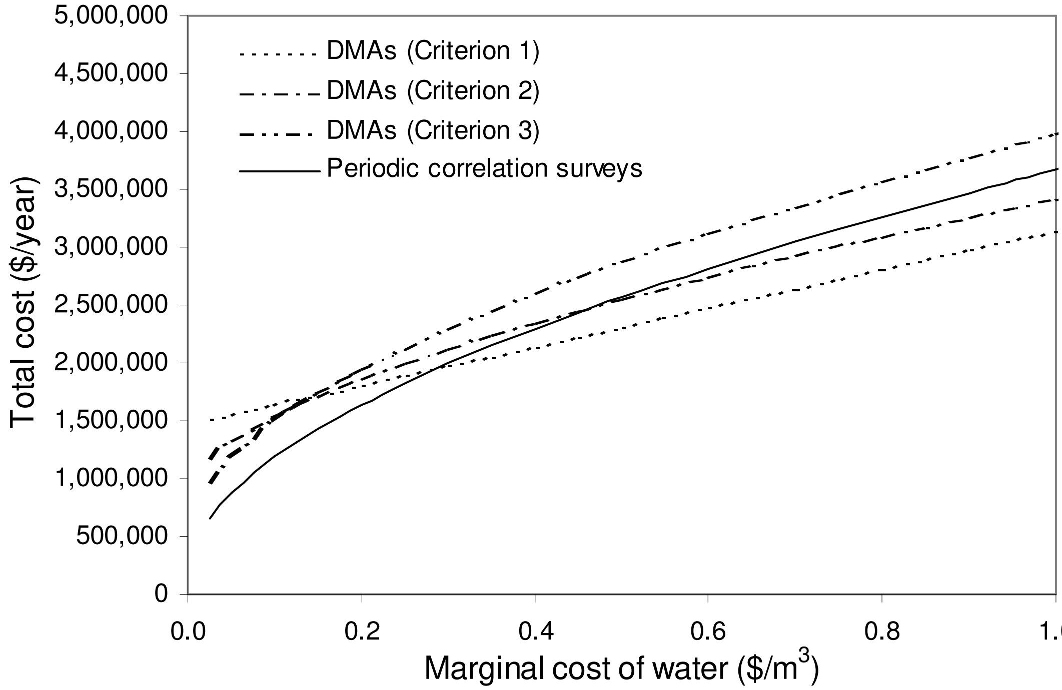
0.6
0.8
1.0
Marginal cost of water ($/m3)

Figure 7



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Total cost ($/year)
0.0
0.2
0.4
0.6
0.8
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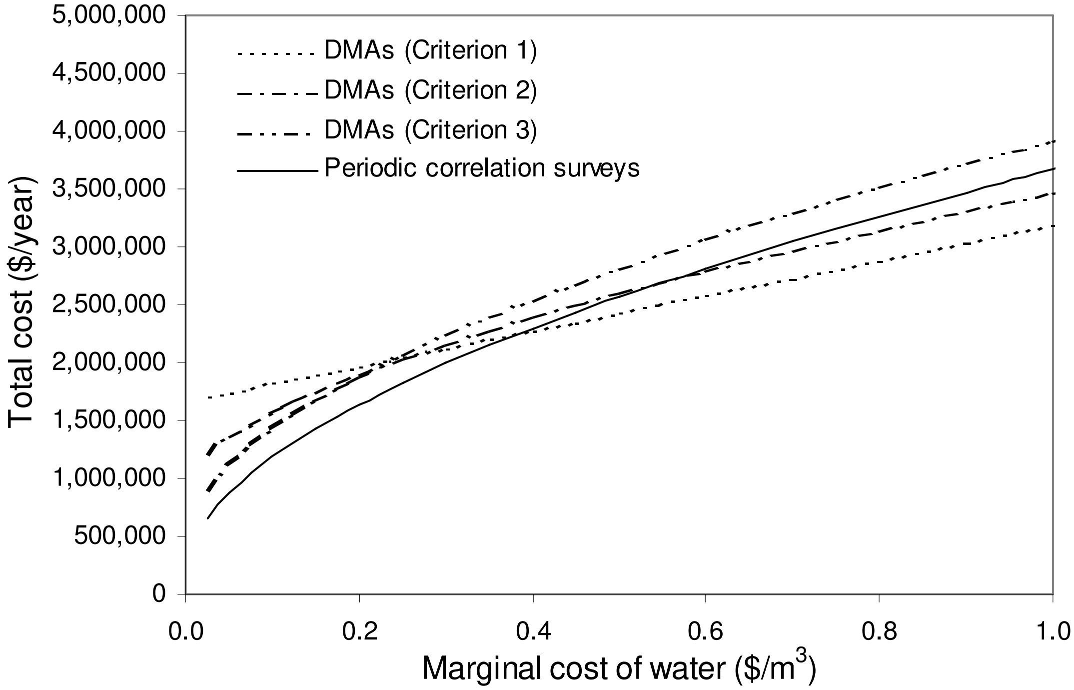
Marginal cost of water ($/m3)
DMAs (Criterion 1)
DMAs (Criterion 2)
DMAs (Criterion 3)
Periodic correlation surveys

Figure 8



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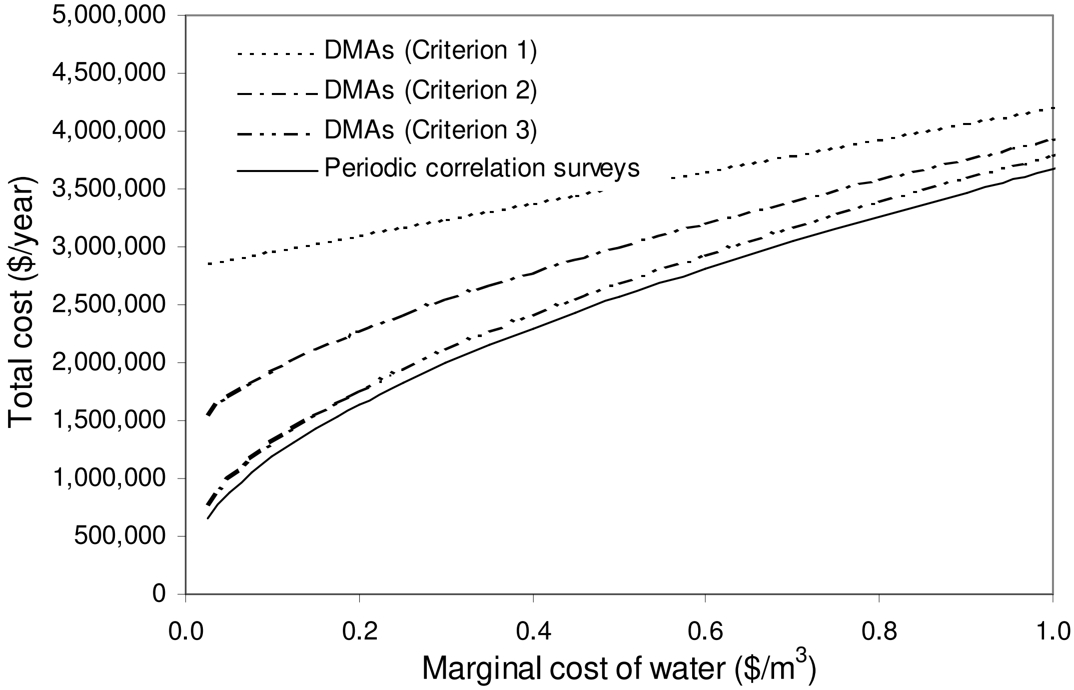
Total cost ($/year)
DMAs (Criterion 1)
DMAs (Criterion 2)
DMAs (Criterion 3)
Periodic correlation surveys
0.0
0.2
0.4
0.6
0.8
1.0
Marginal cost of water ($/m3)

Figure 9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0
Total cost ($/year)
DMAs (Criterion 1)
DMAs (Criterion 2)
DMAs (Criterion 3)
Periodic correlation surveys
0.0
0.2
0.4
0.6
0.8
1.0
Marginal cost of water ($/m3)

Figure 10