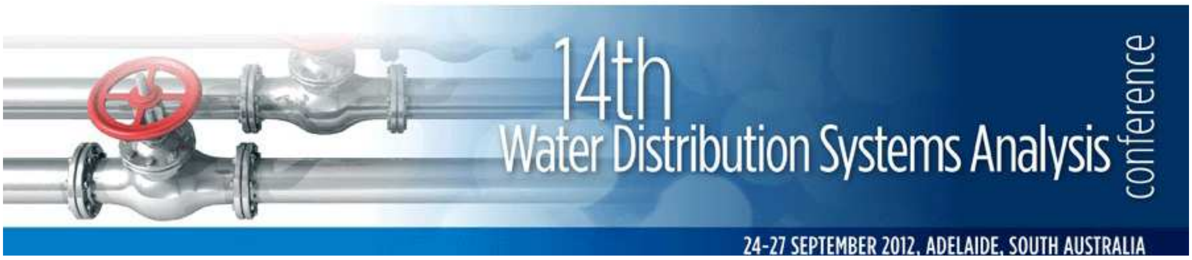


**The Battle of the Water Networks II – Adelaide 2012 (BWN-II)**
**Water Distribution Systems Analysis Conference 2012**
**Adelaide, South Australia, Australia.**
**September 24-27, 2012**
**http://wdsa2012.com/**
**Detailed Problem Description and Rules**
**Release Date January 17, 2012**

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## 1. Introduction

The Battle of the Water Networks (BWN-II) is the fourth in a series of "Battle Competitions" dating back to the Battle of the Water Networks (BWN) in 1985 and more recently the Battle of the Water Sensor Networks (BWSN) in 2006 and the Battle of the Water Calibration Networks (BWCN) in 2010.

The BWN-II calls for teams/individuals from academia, consulting firms, and utilities to propose a design methodology and apply it to a real water distribution system. The results of the BWN-II will be presented at a special session of the upcoming 14th Water Distribution Systems Analysis Symposium in Adelaide, South Australia in September 2012 (http://wdsa2012.com/).

It should be emphasized that none of the BWN-II organizers listed above is taking part in the BWN-II as participants. The organizer’s responsibility is to assemble the design approaches and results, make sure that the BWN-II is objectively assessed, organize the session at the WDSA2012 event, and prepare a journal manuscript (as warranted) to be submitted to the Journal of Water Resources Planning and Management Division, ASCE, to summarize the outcomes and results of the competition.

The rest of this document describes the BWN-II competition rules and framework.

## 2. How to participate

Each participating team/individual must submit by February 17, 2012 an on-line abstract for the WDSA2012 conference (http://wdsa2012.com/) that discusses briefly the proposed design approach (e.g., trial and error with simulation, evolutionary computation, heuristics, etc.). When submitting the abstract the topic area must be identified as "Battle of the Water Networks II" – this will identify your team of authors as participants in the BWN-II. Notifications of accepted/rejected abstracts will be made by March 16, 2012. Each successful team must summarize their final calibration results in a conference paper; these must be uploaded to the WDSA2012 web site by July 3, 2012. All conforming designs will be included in the public presentation of summary results at the conference and will be published as part of the conference proceedings. Selected participants will present their results at a special workshop on the workshop day before the conference.

Authors who wish to submit an abstract before the due date of February 17, 2012 are welcome to do so. The organizers will evaluate abstracts as they are submitted and will notify the authors of acceptance/rejection as soon as possible (upon submission at the conference web site please forward a copy of the submitted abstract to the BWN-II organizers).

Submitted papers describing the final designs from each team should be brief and to the point. It is not necessary to describe the BWN-II background. To allow efficient and fair assessment of contributed results, papers submitted to the BWN-II are asked to include the following sections: Abstract; Introduction (brief); Methodology (whether qualitative or quantitative); Summary of design results for the tested network; Discussion of Results; Conclusions; and References.

**In addition to the submission of the paper through the conference web page, participants are requested to email the BWN-II organisers the conference paper and supporting materials at battle@wdsa2012.com. The supporting materials are: (i) an EPANET (version 2.00.12) *.inp file of the designed network for an Extended Period Simulation (EPS) of 168 hrs (see further details below); and (ii) an Excel file reporting the options adopted in the network and the cost associated should be included. The network EPANET file outlining the system details and the template for the Excel file for reporting the design options can be downloaded from http://wdsa2012.com/.**

The Discussion of Results section should include method-specific information about the design effort (e.g., person-hours, computer type and execution time/memory requirements, scenarios used for verification purposes, etc.). Results submitted with incomplete information may be excluded from the comparison.

## 3. Event schedule

Table 1 lists the schedule for the BWN-II.

Table 1. BWN-II schedule

| | |
|---|---|
| Deadline for submission of abstract | 17 February 2012 |
| Notification of acceptance of abstract for the BWDSA-2012 | 16 March 2012 |
| Final date for upload of submissions | 3 July 2012 |

The competition winner(s) will be declared at the WDSA conference and results will be presented at a special workshop on the workshop day before the conference.

## 4. Problem description

The municipality of D-Town is in need of a design project to cope with the increased water demand of the population and a general increase in the population that has led to a new residential district in the Eastern part of the city (junctions from N1 to N10 in the EPANET file, where the demand pattern assigned is DMA3_pat). To accomplish this task the city has already commissioned the development of a calibrated hydraulic model of the actual network so as to better evaluate its present state and its future behaviour (as outlined above, this file can be downloaded from http://wdsa2012.com/). The network model includes the forecasted demands, the demand patterns and the layout of the new district. It also contains existing pump and tank characteristics and the actual controls of pumps and valves. The model shows that the existing infrastructure is not able to meet the forecasted demand and therefore an upgrade of the network is necessary.

The water utility is interested in minimizing operational and capital costs, but is also interested in reducing the greenhouse emissions, as it is likely that cap and trade policy or a carbon tax will be introduced in the future. Moreover, as the actual network model shows that the water is in the network for long periods of time before reaching the users, the utility is also interested in reducing the water age in the network.

The specific criteria required by the water utility to assess the project design and the design options are outlined below.

### 4.1 Design Requirements

#### 4.1.1 Costs

The water utility desires a low operational and capital cost design. In particular, the operational costs are a result of pump operations only, while capital costs are associated with the pipe and tank material and construction, and any upgrades of the existing pumping stations.

As capital and operational costs occur at different time points during the lifetime of the project, annual costs are provided for the new possible components of the network in the section *Design Options*. These costs take into account the lifetime of each specific component and the discount rate. Consequently, the total cost requiring minimization is the sum of the annual capital costs and of the annual cost of pumping operations.

To account for the annual cost of pumping operations, the weekly cost of the pump power has to be computed. This cost is then multiplied by the number of weeks in a year (52) and, to account for the demand variability throughout the year, this number is divided by the peak-day factor of 1.3. (The variability of the electric tariff, of the demand, and of any other design variable, during the year and the lifetime of the project is not considered.)

#### 4.1.2 Water age

The water age is considered by the utility to be too high in some parts of the network, and the utility desires that the water age within the project design to be kept to a minimum. The utility has defined the measure of network water age as follows:

$$WA_{net} = \frac{\sum_{i=1}^{N_{junc}} \sum_{j=1}^{N_{time}} k_{ij} Q_{dem,ij} WA_{ij}}{\sum_{i=1}^{N_{junc}} \sum_{j=1}^{N_{time}} Q_{dem,ij}}$$

where $WA_{net}$ is the network water age (hours), $WA_{ij}$ is the water age (hours) at junction $i$ (note: tanks/reservoirs excluded) at time $t_j$, $k_{ij}$ is the variable defined as follows:

$$k_{ij} = \begin{cases} 1 \quad , \quad WA_{ij} \geq WA_{th} \\ 0 \quad , \quad WA_{ij} < WA_{th} \end{cases}$$

where $WA_{th}$ is the water age threshold equal to 48 hours (for this competition), $Q_{dem,ij}$ is the demand at junction $i$ and time $t_j$, $i$ is the junction index ($i$=1, 2, ..., $N_{junc}$), $j$ is the time index ($j$=0,1,...,$N_{time}$), $t_j$ is the simulation time $t_j=j\Delta t$, $\Delta t$ is the time step equal to 1 hour (resulting in all water age and demand variables to be computed only on the hour), $N_{junc}$ is the number of system junctions and $N_{time}$ is the number of simulation time steps (equal to 168 as the extended period simulation time is one week). $WA_{net}$ should be assessed during normal system operation only.

Note that the above network water age measure considers water age above the threshold only which is compatible with AWWA/EPA guidelines. It also considers water age at non-zero demand nodes only and gives more weight to nodes with larger demands (implying a risk type indicator). Finally, the above network water age measure takes into account water age at junctions only (the age of water in a tank is taken into account indirectly, via corresponding demand junctions supplied from that tank).

### 4.1.3 Greenhouse gas emissions

The water utility foresees that the introduction of a carbon tax, or a cap and trade policy, is likely to occur in the near future. Hence they would like to minimize the greenhouse gas (GHG) emissions associated with the design.

From the water utility's perspective, GHGs can be subdivided into (a) capital GHG emissions (associated with the energy used to produce, transport and install pipes, tanks and pumps), and (b) operational GHG emissions (resulting from the operation of pumps drawing their electricity from fossil fuel sources). In particular, the water utility wants to minimize the annualized total GHG emissions, which are considered as the sum of the annualised capital GHG emissions, and of the annual operational GHG emissions.

The capital component of the annual GHG emission is estimated using Table 2, which represents the annualised $CO_2$-equivalent emissions to manufacture a meter of a pipe.

Table 2. Annual $CO_2$ equivalent emissions to manufacture the pipes.

| **Diameter (mm)** | **Annualised EE (kg-$CO_2$-e/m/year)** |
|---|---|
| 102 | 5.90 |
| 152 | 9.71 |
| 203 | 13.94 |
| 254 | 18.43 |
| 305 | 23.16 |
| 356 | 28.09 |
| 406 | 33.09 |
| 457 | 38.35 |
| 508 | 43.76 |
| 610 | 54.99 |
| 711 | 66.57 |
| 762 | 72.58 |

Annual operational GHG emissions are computed as the product of total annual pump energy consumption in the system under normal operation (see section 4.1.1) and the emission factor of 1.04 kg-$CO_2$-e/kWh.

*Additional information*:
As in the case of annual capital costs, values shown in Table 2 already take into account the lifetime of the components and a discount factor. The discount factor here has been chosen to be 0% as recommended by the Intergovernmental Panel in Climate Change (IPCC). Emissions to manufacture pumps and tanks will be neglected due to unavailability of accurate data at this stage.

The emission factor to convert the energy into a carbon equivalent (1.04 kg-$CO_2$-e/kWh) is assumed to be constant. Although the emission factor changes according to the sources used to produce the energy (and therefore according to the time of the day, the period of the year and region, and it will also change in the future as the mix of renewable energy sources changes), a constant value is considered for simplicity.

### 4.1.4 System performance under normal operation

The water utility requires that every demand node of the network has water delivered to it with adequate pressure. Nodes without demand only have the requirement of a minimum pressure above zero. The minimum pressure required for nodes with demands is 25 m.

An additional requirement of the water utility is that at the end of the extended period simulation (1 week) each tank has to have at least the same volume of water it had at the beginning of the simulation. Note that this initial volume has to be set equal to half the volume of the tank (see Figure 1). Moreover, during normal operation, tanks are not allowed to stay empty.

In the EPANET2 input file, tanks are described by 6 values: diameter, elevation, minimum level, maximum level and initial level as shown in Figure 1. In EPANET2 the tank level is forced to stay between the minimum and the maximum tank level, so that the volume between the tank bottom elevation and the minimum level represents a reserve volume. If a simulation requires water from a tank that is already at its minimum level, EPANET2 disconnects the tank so that the tank level will remain at its minimum value.

The requirement of tanks not staying empty can be therefore translated in EPANET2 by checking that the tank water level (shown in the field “Pressure” in the tank property) is not equal to the minimum level for two or more consecutive time steps. In the input file provided, all tanks have a minimum level set equal to 0 m and hence the whole volume of the tank can be used. The tank minimum level can be changed as desired by the participants.

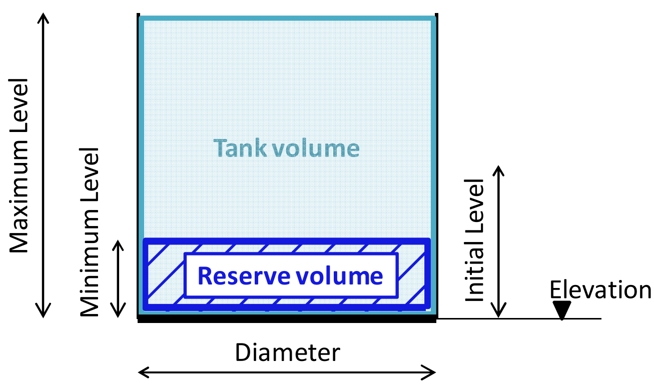


Figure 1. Tank description in EPANET2.

### 4.1.5 System performance under power failure operation

The provision of a reliable water supply is critical for the proposed network design. The water utility's major concern about failing to provide adequate supply is exclusively due to the possibility of a power outage. This is because a power outage will result in a shutdown of the network's pumps (except the pumps powered by the backup diesel generators), leading to an emptying of the supply tanks and the inadequate provision of pressure at the network nodes.

The water utility has characterised the design power failure event as being a two hour power outage. In the event of a power outage, the system is still expected to provide a minimum pressure of 25 m to nodes with a non-zero water demand; otherwise the minimum pressure required is zero. Moreover, tanks are not allowed to stay empty. Note that during power failure, tanks are allowed to operate below their minimum level if this value has been set larger than zero (that is, the reserve volume can be used during the power failure operation).

The power outage can occur at any time during the simulation week. Therefore, for a power failure scenario starting at time *t*, initial tank levels will be those determined from the normal operation conditions at time *t*, and the demands for the two hour duration of the power failure will be consistent with those specified in the EPANET2 input file provided for this time duration.

Please note that, in case of a power failure, the network performance will be assessed only for the two hours of power outage. Note also that the network performance can be improved by increasing the network storage volume and/or by introducing diesel generators (see Sections 4.2.2, and 4.2.4). If one or more diesel generators are selected, the pumps connected to them will be assumed constantly switched "ON" for two hours, i.e. for the full duration of power outage. Clearly, if a pump in not connected to a power generator, it will be switched "OFF" for those two hours.

## 4.2 Design Options

### 4.2.1 Pipes

Pipe diameter options and costs for the network expansion are given in Table 3. The costs shown are inclusive of pipe construction, transport and installation.

According to the water utility, pipes can also be placed in parallel to existing pipes (pipe IDs starting with the letter P in the EPANET2 input file). However, as this implies the disruption and reconstruction of pavement roads, the cost of duplicating existing pipes is given by the costs within Table, but with an additional cost premium of 20%.

Another requirement of the water utility is that none of the existing pipes can be decommissioned, with the exception of pipes with IDs equal to 1 and 2 in the EPANET2 input file. In fact, the water utility is not sure if it is more effective to link the new zone to the DMA3 or to the DMA2. Therefore one of the two pipes that link the new zone to the existing districts (pipes 1 and 2) can be omitted (a diameter option can be zero, meaning that the pipe is not built).The Hazen-Williams coefficient for every diameter is equal to 120.

Table 3. Pipe annual costs.

| Diameter | Annual Cost New Pipe | Annual Cost Parall Pipe |
|---|---|---|
| (mm) | ($/m/yr) | ($/m/yr) |
| 102 | 8.31 | 9.97 |
| 152 | 10.10 | 12.10 |
| 203 | 12.10 | 14.49 |
| 254 | 12.96 | 15.55 |
| 305 | 15.22 | 18.28 |
| 356 | 16.62 | 19.94 |
| 406 | 19.41 | 23.26 |
| 457 | 22.20 | 26.65 |
| 508 | 24.66 | 29.58 |
| 610 | 35.69 | 42.80 |
| 711 | 40.08 | 48.12 |
| 762 | 42.60 | 51.11 |

### 4.2.2 Tanks

Because of the increased demands, the water utility is also allowing for the addition of new tanks, but only adjacent to the existing tanks, where the water utility already owns sufficient land. New tanks are assumed to have the same height and bottom elevation as the existing adjacent tanks (because the water utility does not want to introduce new valves to control the system). All new tanks are cylindrical and come in pre-specified standard sizes shown in Table 4, together with associated annualised costs. The construction of non-standard tanks is discarded by the water utility because they are regarded as being too expensive.

Table 4. Tank annual costs.

| Volume | Annual Cost |
|---|---|
| ($m^3$) | (1000$/yr) |
| 500 | 14.02 |
| 1000 | 30.64 |
| 2000 | 61.21 |
| 3750 | 87.46 |
| 5000 | 122.42 |
| 10000 | 174.93 |

Note that the annual costs shown in Table 4 already include the connectivity costs to link the new tanks to the network. Therefore, the addition of new tanks can be modelled in EPANET simply by increasing the tank diameters so that the resulting volume is equal to the existing tank volume plus the new tank volume. Note also that all the constraints on operational levels of new and existing tanks have been already given in sections 4.1.4 and 4.1.5.

### 4.2.3 Pumps

Existing pump systems can be upgraded by replacing existing pumps with larger pumps or by adding new pumps to the existing pumping stations in parallel to the existing pumps. No additional pumping stations or boosters can be placed into the network as the water utility does not have any available location for these new components. Pump efficiency is assumed constant: existing pumps have an efficiency of 65%, while new pumps have an efficiency equal to 75%. Pump curves for available pumps are given in the Appendix in Tables A1-A10. Table 5 shows the annual costs associated with each pump.

Table 5. Pump annual costs.

| Pump Model | Maximum Power | Annual Cost |
|---|---|---|
| | kW | ($/yr) |
| 8 | 45.24 | 4133 |
| 9 | 31.67 | 3563 |
| 10 | 49.76 | 4339 |
| 11 | 22.62 | 3225 |
| 8a | 22.62 | 3225 |
| 10a | 24.88 | 3307 |
| 11a | 11.31 | 2850 |
| 8b | 54.28 | 4554 |
| 9b | 38.00 | 3820 |
| 10b | 59.71 | 4823 |

### 4.2.4 Diesel generators

To improve the network's performances during power outage, the water utility allows for the purchasing and installation of diesel generators to be located within the existing pump stations. Note that more than one diesel generator can be inserted in the same pumping station. The costs given in Table 6 include generator costs, and related transport/installation costs. No costs associated with the land property needs to be considered.

Table 6. Diesel generator annual cost.

| Power | Annual Cost |
|---|---|
| (kW) | (1000$/yr) |
| 50 | 9.45 |
| 100 | 10.56 |
| 200 | 11.63 |
| 300 | 15.00 |
| 400 | 16.78 |
| 800 | 25.74 |
| 1300 | 40.67 |
| 2500 | 62.29 |

Diesel generators should be installed to accommodate the total rated power of all pumps backed up by the generators, where the rated power of a pump corresponds to the maximum pump power given in Table 5. Energy consumption and GHG emissions associated with the use of diesel generators are considered negligible and hence should not be computed in the evaluation of performances related to the cost and the GHG emissions.

### 4.2.5 Valves

The water utility is not considering introducing any additional operational valves in the system. The pressure reducing valves currently present in the system are to be maintained (i.e. cannot be removed), but their operational settings can be modified if deemed necessary. However, their operational setting cannot be set above 60m. The only exception to the insertion of new valves is the possible installation of the PRV valve N15 in the new pipe 2 linking the new zone to the DMA3. If this valve is installed, its annualised cost should be determined from Table 7 for the pipe 2 diameter.

Table 7. PRV annual cost.

| **Diameter (mm)** | **Annual Cost ($/yr)** |
|---|---|
| 102 | 323 |
| 152 | 529 |
| 203 | 779 |
| 254 | 1113 |
| 305 | 1892 |
| 356 | 2282 |
| 406 | 4063 |
| 457 | 4452 |
| 508 | 4564 |
| 610 | 5287 |
| 711 | 6122 |
| 762 | 6790 |

In all cases, modified settings of PRVs are assumed constant during simulation period, nor are they allowed to depend on the status of other network components). Note that the setting of existing PRVs and of the PRV N15 does not introduce any additional cost.

### 4.2.6 Pump controls

The existing pump controls shown in the EPANET input file provided can be changed without incurring additional costs. The same applies to changing the pump type control, i.e. switching from pumps controlled by the corresponding threshold tank levels to time controlled pumps or vice versa.

### 4.3 Electricity Tariff

The electricity tariff is shown in Table 8 where the energy prices are shown in cents/kWh. For example, the price applied on Monday is 6.72 cents/kWh from 6:00 am to 7:00 am, while it is 10.94 cents/kWh from 7:00 am to 8 am.

Table 8. Electricity tariff.

| Hour/Day | Mon | Tue | Wed | Thu | Fri | Sat | Sun |
|---|---|---|---|---|---|---|---|
| **0** | 6.72 | 6.72 | 6.72 | 6.72 | 6.72 | 6.72 | 6.72 |
| **1** | 6.72 | 6.72 | 6.72 | 6.72 | 6.72 | 6.72 | 6.72 |
| **2** | 6.72 | 6.72 | 6.72 | 6.72 | 6.72 | 6.72 | 6.72 |
| **3** | 6.72 | 6.72 | 6.72 | 6.72 | 6.72 | 6.72 | 6.72 |
| **4** | 6.72 | 6.72 | 6.72 | 6.72 | 6.72 | 6.72 | 6.72 |
| **5** | 6.72 | 6.72 | 6.72 | 6.72 | 6.72 | 6.72 | 6.72 |
| **6** | 6.72 | 6.72 | 6.72 | 6.72 | 6.72 | 6.72 | 6.72 |
| **7** | 10.94 | 10.94 | 10.94 | 10.94 | 10.94 | 10.94 | 6.72 |
| **8** | 10.94 | 10.94 | 10.94 | 10.94 | 10.94 | 10.94 | 6.72 |
| **9** | 10.94 | 10.94 | 10.94 | 10.94 | 10.94 | 10.94 | 6.72 |
| **10** | 27.68 | 27.68 | 27.68 | 27.68 | 27.68 | 10.94 | 6.72 |
| **11** | 27.68 | 27.68 | 27.68 | 27.68 | 27.68 | 10.94 | 6.72 |
| **12** | 27.68 | 27.68 | 27.68 | 27.68 | 27.68 | 10.94 | 6.72 |
| **13** | 27.68 | 27.68 | 27.68 | 27.68 | 27.68 | 10.94 | 6.72 |
| **14** | 27.68 | 27.68 | 27.68 | 27.68 | 27.68 | 10.94 | 6.72 |
| **15** | 27.68 | 27.68 | 27.68 | 27.68 | 27.68 | 10.94 | 6.72 |
| **16** | 27.68 | 27.68 | 27.68 | 27.68 | 27.68 | 10.94 | 6.72 |
| **17** | 10.94 | 10.94 | 10.94 | 10.94 | 10.94 | 10.94 | 10.94 |
| **18** | 10.94 | 10.94 | 10.94 | 10.94 | 10.94 | 10.94 | 10.94 |
| **19** | 10.94 | 10.94 | 10.94 | 10.94 | 10.94 | 10.94 | 10.94 |
| **20** | 10.94 | 10.94 | 10.94 | 10.94 | 10.94 | 6.72 | 10.94 |
| **21** | 6.72 | 6.72 | 6.72 | 6.72 | 6.72 | 6.72 | 6.72 |
| **22** | 6.72 | 6.72 | 6.72 | 6.72 | 6.72 | 6.72 | 6.72 |
| **23** | 6.72 | 6.72 | 6.72 | 6.72 | 6.72 | 6.72 | 6.72 |

## 5. Design evaluation

Each participant is required to submit one solution only - regardless of the methodology used. The solutions received will be ranked for each of the performance criteria of cost, water age, and GHG emissions and assigned a score depending on this ranking (given compliance to all performance criteria for both normal and power outage operations). The solution with the highest overall rank will be selected as the winner. Although they will be omitted from consideration, it is possible to describe other solutions that the participants have obtained in the paper.

Note that, as outlined earlier, to be eligible to participate, participants are required to submit: (i) a paper describing the approach adopted; (ii) the EPANET input file (*.inp) of the network solution selected (set up for the single week extended period simulation); and (iii) the completed Excel file outlining the design decisions made by the participants. Note also that the selected solution will be independently checked and evaluated using the hydraulic solver EPANET2 to verify the authors' results.

## 6. Questions about the competition

Contact: Aaron Zecchin (azecchin@civeng.adelaide.edu.au) and Avi Ostfeld (ostfeld@tx.technion.ac.il). Dr. Zecchin and Prof. Avi Ostfeld will direct questions to appropriate members of the subcommittee who have organized this competition.

**APPENDIX:**

Available pump curves are shown in Tables A1-A10, where curves 8, 9, 10, 11 correspond to pumps currently operating in the network. It has to be noted that the pump curves cover the whole range of flows (from flow equal to 0 to the flow corresponding to zero head): the head-flow equation is also given in the tables where the equations are consistent with the three point curve fitting used by EPANET2.

Table A1. Pump curve 8: $H$=70.00-0.07731·$Q^{1.36}$.

| **Pump Model** | **Flow (L/s)** | **Head (m)** |
|---|---|---|
| 8 | 0 | 70 |
| 8 | 60 | 50 |
| 8 | 100 | 30 |

Table A2. Pump curve 9: $H$=90.00-0.01331·$Q^{2.15}$.

| **Pump Model** | **Flow (L/s)** | **Head (m)** |
|---|---|---|
| 9 | 0 | 90 |
| 9 | 30 | 70 |
| 9 | 50 | 30 |

Table A3. Pump curve 10: $H$=120.00-0.001477·$Q^{2.59}$.

| **Pump Model** | **Flow (L/s)** | **Head (m)** |
|---|---|---|
| 10 | 0 | 120 |
| 10 | 30 | 110 |
| 10 | 70 | 30 |

Table A4. Pump curve 11: $H$=90.00-0.01104·$Q^{2.41}$.

| **Pump Model** | **Flow (L/s)** | **Head (m)** |
|---|---|---|
| 11 | 0 | 90 |
| 11 | 30 | 50 |
| 11 | 40 | 10 |

Table A5. Pump curve 8a: $H$=70.00-0.198·$Q^{1.36}$.

| **Pump Model** | **Flow (L/s)** | **Head (m)** |
|---|---|---|
| 8a | 0 | 70 |
| 8a | 30 | 50 |
| 8a | 50 | 30 |

Table A6. Pump curve 10a: $H$=120.00-0.008915·$Q^{2.59}$.

| **Pump Model** | **Flow (L/s)** | **Head (m)** |
|---|---|---|
| 10a | 0 | 120 |
| 10a | 15 | 110 |
| 10a | 35 | 30 |

Table A7. Pump curve 11a: $H$=90.00-0.05866·$Q^{2.41}$.

| **Pump Model** | **Flow (L/s)** | **Head (m)** |
|---|---|---|
| 11a | 0 | 90 |
| 11a | 15 | 50 |
| 11a | 20 | 10 |

Table A8. Pump curve 8b: $H$=84.00-0.09277·$Q^{1.36}$.

| **Pump Model** | **Flow (L/s)** | **Head (m)** |
|---|---|---|
| 8b | 0 | 84 |
| 8b | 60 | 60 |
| 8b | 100 | 36 |

Table A9. Pump curve 9b: $H$=108.00-0.01597·$Q^{2.15}$.

| **Pump Model** | **Flow (L/s)** | **Head (m)** |
|---|---|---|
| 9b | 0 | 108 |
| 9b | 30 | 84 |
| 9b | 50 | 36 |

Table A10. Pump curve 10b: $H$=144.00-0.001773·$Q^{2.59}$.

| **Pump Model** | **Flow (L/s)** | **Head (m)** |
|---|---|---|
| 10b | 0 | 144 |
| 10b | 30 | 132 |
| 10b | 70 | 36 |