Using a new pressure index for water distribution systems upgradation improvement evaluation
Savalan Pour Akbarkhiavi and Monzur Alam Imteaz

ABSTRACT

Potable water distribution systems (WDS) require upgrade strategies based on a pre-defined time interval which is identified by the responsible water authorities. The main goal of a potable water system upgrade is maintaining the standard and acceptable level of service after the occurrence of increases in the serviced population, asset ageing, and/or development of the serviced area. Defining the level of service varies by location according to the codes and regulations adopted by the water authority. In general, two main factors are notable in planning of WDS upgrade strategies: (1) the 'level of service' and (2) the 'upgrade cost'. In the presented paper, a new index has been introduced to evaluate the level of service for WDS from a pressure point of view. The new index that is presented in this paper is named the 'Pressure Index (PI)', and incorporates a number of water connections for five different pressure regimes. As a case study, three existing water network systems in the Castlemaine township area, located in central Victoria, Australia, have been investigated and the relationship between the 'upgrade costs' and improvement in PI factors is presented.

Key words | level of service, network planning, pressure index, upgrade cost, water distribution system

INTRODUCTION

Cities, townships and other inhabited areas around the world are currently facing considerable problems to manage urban development and population growth. Under such conditions, water distribution system (WDS) planning is playing a crucial role to improve the effectiveness of investments made in the urban water systems. Proper water asset planning should include a comprehensive and rigorous identification of all options to meet the defined service levels, including options based on non-asset solutions. Planning should be an iterative process which attempts to balance service needs with infrastructure, operation and maintenance, capital expenditure and the environment.

The most expensive element of a water supply system is the distribution network. Deficiency in the distribution network due to ageing and stress will cause significant operation and maintenance costs, water leakage, decrease in the quality of service, and drop in the quality of water supplied (Kleiner et al. 1998). The US Environmental Protection Agency (USEPA) research estimated the budget necessities for upgrading WDS and transmission systems in the United States to be US$77 billion, approximately, over the next 20 years (Davies et al. 1997). Canadian Water and Waste-water Association (CWWA) estimates that CAN$11.5 billion will be needed just for water main upgrades over the next 15 years (CWWA 1997).

Effective WDS planning outcomes can only result from rigorous analysis, the application of strategic thinking skills and the adoption of an integrated approach to urban water planning which considers, where appropriate, water supply, sewerage and management of stormwater as a single system (DELWP 2015).

Due to the random nature of future water needs, customer pressure requirements and pipe roughness changing
with time, the estimation of the reliability of WDS for the future works involves some uncertainty (Bao & Mays 1990). Some other studies demonstrate that uncertainties in future water demands, pressure requirements, and pipe roughness can have significant effects on the optimal network design and cost (Lansey et al. 1989). Recent researchers focused on how WDS can be made more resilient and adaptable, thus reducing their vulnerability to future changes (Farmani & Butler 2015). In the WDS pressure management field, recent studies tend to emphasize the influence of the pressure management on the pressure/leakage relationships (Gomes et al. 2013).

The current common approach for WDS asset planning is utilizing advanced and sophisticated hydraulic models (Bush & Uber 1998) and it has been proven that these models can be used for planning purposes by simulating the existing assets and applying future scenario demands (Alperovits & Shamir 1977). There are some issues involved in using hydraulic models such as:

- necessity in understanding complicated hydraulic concepts related to water networks;
- spending significant amounts of time and resources on establishing existing and future scenarios; and
- the notable market price of existing hydraulic model platforms (AWWA 1974).

In addition to the mentioned issues, the desired outcomes of the modelling work and the details are required to be established before commencing the process. This is where it is important that operational staff is involved in the construction and analysis of the network model.

A successful network modelling requires the investment of time by experienced staff to interpret the results of the modelling. Model outputs should be verified against actual system performance and are needed to be calibrated on the basis of field measurements such as pipe flows, pipe pressures and storage tank levels. The whole process is costly and time consuming for the responsible water authority. The main aim of upgrading a water system is maintaining the acceptable level of service after increases in serviced population/area and the ageing of assets. In general, WDS upgrade works are constrained by budget allocation and every upgrade is a cost-intensive exercise (Alonso et al. 2000). Therefore, on one side the critical customer-oriented parameter is ‘level of service’ and on the other side the critical factor for the authority is ‘upgradation cost’ (Eiger et al. 1994).

The first step in any comprehensive approach in proper asset planning in WDS is obtaining a forecast of a system’s future water demands based on a rational population projection (DELWP 2013). The next step in a successful typical WDS asset plan is the inclusion of a feasible and efficient set of options for catering the predicted demand based on the defined level of service criteria (Tospornsampan et al. 2007). Until now, very limited studies have been conducted to investigate the relationship between maintaining the accepted ‘level of service’ and the associated ‘upgrade cost’. Currently, there is no standard factor and/or index to compare a particular upgrade option with another potential option from the level of service perspective.

**METHODOLOGY**

The primary objective of this research is the development of a tool to identify the relationship between level of service for a WDS and the cost of essential water asset upgrades to provide an acceptable level of service range.

Therefore, the major aims of the study are to:

- introduce a new factor to evaluate the performance of a typical WDS from a pressure point of view;
- propose a method to compare different potential upgrade options from a level of service perspective;
- identify an acceptable range for level of service from the connection pressures perspective;
- develop a tool to estimate the upgrade cost associated with the defined acceptable level of service range; and
- conduct case studies for existing WDS.

To cover the above aims, a new factor in WDS has been defined to address the existing gap in the evaluation of ‘level of service’ in a WDS. A survey has been conducted among a number of Victorian water authorities to obtain their opinions on proposed level of service factors and their weightings.
Three Victorian townships have been examined as a case study of existing WDS for the newly defined factors. Advanced hydraulic models have been built and utilized to simulate the WDS case studies. InfoWorks WS has been used in this research as the main modelling and calibration platform for the models. In parallel to the water network model built for the three case study townships, a flow and pressure monitoring activity was done to have a comparison between the results gained from the InfoWorks WS model and the real flows and pressures in the system. By using the flow and pressure monitoring data, the hydraulic models were calibrated to get the real diurnal profiles of the water consumption for each town and adjustments were made to the pipes roughness coefficient to replicate the measured pressured in the hydraulic model. The water loss and leakage were calculated by comparing the billing data with the produced water from the main water sources.

Future water demand, including scaling factors, were adopted based on actual system performance, historical records and a consideration of future demand. The components of water were calculated as mentioned and assigned to the model. The approved upgrade options by the responsible water authority (Coliban Water) were developed, new assets and upgrades were modelled and the achieved results were examined. The associated cost of each approved upgrade option by Coliban Water was used to identify the relationship between the upgrade costs and the improvement in the level of service.

**STUDY AREA**

As a case study, the water supply systems of three townships in the vicinity of Castlemaine, in Victoria, Australia, have been studied. The water supply systems include the townships of Castlemaine, Chewton, Campbells Creek, Fryerstown, Guildford, Yapeen, Harcourt, Maldon and Newstead. Figure 1 shows the locality of the case study area.

The systems studied serve a total population of 13,350 residents as well as commercial and industrial customers. Approximately 7,050 properties are connected to the system (Coliban Water 2010).

A schematic of the comprehensive Castlemaine WDS is provided in Figure 2.

In this study, the adopted growth in water demand was based on residential and non-residential growth data provided by the related water authority (Coliban Water 2010).

The developed planning strategy was focused on the following:

- Augmentation requirements to satisfy current and future demands and achieve adequate security of supply to all areas for the proposed long term growth projections.
- Operations plans for the enhancement of the current Castlemaine water network system. In broad terms, these plans specify the future configuration of the networks, the objectives for managing levels in the various water storages and ensuring water quality is achieved throughout the system.

![Figure 1](https://iwaponline.com/ws/article-pdf/16/5/1339/411310/ws016051339.pdf)
The development of the augmentation strategy that has been used in this research is based on the following:

- A calibrated water supply hydraulic model of the comprehensive Castlemaine water network.
- Water supply demands imposed upon the hydraulic model that reflect the forecast growth and demand policies anticipated for this system.
- System deficiencies have been identified via hydraulic model runs and comparing the outputs from these against the water authority’s adopted supply standards.
- Development of potential augmentation and upgrade options has been driven by addressing and mitigating system deficiencies and improving the security of supply throughout the system.

The focus of this paper is on the following three systems in the comprehensive Castlemaine WDS area:

- The Castlemaine township
- The Harcourt township
- The Campbells Creek township.

**DATA**

The 2010 demands defined by the water authority were used and allocated in the developed InfoWorks WS baseline model peak day scenario as the baseline.

The total average day demand was forecasted to increase by 41% from current levels to 2038, which was defined as the ultimate target year of the network planning. The total peak day demand (PDD) was estimated to be approximately 2.7 times the average day demand based on data provided by the water authority. Based on the respective water authority’s approved demand, the adopted peak day residential demands were:

- 1,825 L/connection/day for urban areas;
- 2,320 L/connection/day for small towns.

These demands have been allocated to all new connections based on the demand category that has been allocated to adjacent demand nodes. Average day demands have also been allocated to all new connections based on the average demands for each pressure zone. The general approach to
developing solutions for the Castlemaine upgrade strategy for the purpose of this research was to ensure that the system could satisfactorily meet the minimum levels of service and satisfy forecasts for current and future demands, whilst achieving adequate security of supply for all the connections. This has involved the replacement of the aged and overstressed pipes to reduce head loss to acceptable levels and improve pressures in the downstream areas, and upsizing existing storages where they were assessed to be undersized based on future growth in demand. Where pipe augmentations were deemed inadequate to maintain >20 m pressure, options to re-zone problem areas or re-configure supply to enable feed from a higher grade system have been considered (Latchoomun et al. 2015). Although group B connections’ pressures are acceptable from hydraulic point of view, it has been proven that from a pressure management point of view the leakages and water losses will be higher in the connections with >50 m pressures (Latchoomun et al. 2015).

The water authorities in Australia generally want to have a minimum pressure of 20 m at all times in their water supply systems, but sometimes due to remoteness of the demand points from water source or some topographic limitations, they do have ‘supply by agreement’ contract by customers. It means just supplying water to those customers and not supplying 20 m minimum pressures. This group of connections has been defined as group D in the above classification. Group E belongs to the connections which experience a minimum pressure of 5 m or less and never experience negative pressure. This category is not acceptable for water authorities but in rare situations can be adopted. All connections which experience any negative pressure during a 24 h water cycle in water systems have been classified as group F.

Currently, there are some tools (i.e. i2O, flow-works, Mike Urban, HydraliCad), which can evaluate a water network from its pressures point of view. The current tools, parameter or factors to measure the efficiency of a water distribution network from the magnitude of its pressure point of view are focused to provide the judgment on hydraulic performance of the concerned water network based on the local codes and standards. Currently, the common methodology to compare the improvement of a water network before and after an upgradation implementation is the modelling of all scenarios and performing the results comparisons. Therefore, convincing anecdotal information is limited to hydraulic results comparisons for any proposed budget to be invested on a particular network to bring the pressures back to the standard level of service after increase in demand. Although there are some conventional definitions for ‘standard level of service’ regarding pressures in WDS, these definitions normally define the minimum and maximum acceptable extremes of the water systems pressure regime, which have been discussed in the previous sections.

A new factor proposed in this research is to address the above issues. The new factor will be used to evaluate WDS from connections’ pressures point of view before and after implementation of the proposed upgrade strategies. The proposed index is a dimensionless factor and has been named

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**A NEW PRESSURE INDEX**

The current research has been undertaken to study a PDD scenario and, therefore, the developed hydraulic model covered a 24 h cycle of a hot summer day. From a pressures point of view, each water connection in a WDS can be categorized into one or more of the following options:

(A) Customers with maximum pressure more than 80 m in a 24 h cycle in one water system.
(B) Customers with maximum pressure less than 80 m and more than 50 m in a 24 h cycle in water system.
(C) Customers with pressures less than 50 m and more than 20 m in a 24 h cycle in one water system.
(D) Customers with minimum pressure less than 20 m and more than 5 m in a 24 h cycle in water system.
(E) Customers with minimum pressure less than 5 m and more than 0 m in a 24 h cycle in one water system.
(F) Customers with minimum pressure less than 0 m in a 24 h cycle in one water system.

Among the above classification, water authorities in Australia plan to achieve group C and avoid the rest (WSAA 2004). The connections in group A are more likely to cause a burst incident in water systems. Many studies have shown that the high pressure water systems have a higher chance of pipe burst and leakage during the operation life of the system. A recently undertaken case study has shown the relationship between the frequencies of bursts in a district metered area (DMA) and the water pipe pressures in the water system (Latchoomun et al. 2015).
the Pressure Index (PI). Monitoring the PI variations before and after a proposed upgrade strategy will provide a tool to assess the relationship between the improvements of the network performance and the associated cost for the improvement. The study of the PI for approved upgrade strategies in the case study town’s water supply system will identify the acceptable range of PI after proper upgrade strategies. As discussed previously, one of the main criteria in evaluation of the level of service in a WDS is the pressures which occur in a 24 h water use cycle.

Therefore, PI is introduced via the following equation in this research:

\[
PI = N_t \left( \frac{\alpha \left( N_{-\infty} \right) + \beta \left( N_{0} \right) + \gamma \left( N_{5/20} \right) + \delta \left( N_{80/50} \right) + \lambda \left( N_{80/\infty} \right)}{\left( N_{80/\infty} \right)} \right) + N_t
\]

where \( N_t \): Number of total customers in the study system

- \( N_{-\infty} \): Number of customers experience maximum pressure greater than 80 m (Group A)
- \( N_{80/50} \): Number of customers experience maximum pressure less than 80 m and greater than 50 m (Group B)
- \( N_{20/5} \): Number of customers experience minimum pressure less than 20 m and greater than 5 m (Group D)
- \( N_{5/0} \): Number of customers experience minimum pressure less than 5 m and greater than 0 m (Group E)
- \( N_{0/\infty} \): Number of customers experience minimum pressure less than 0 m (Group F)

\( \alpha, \beta, \gamma, \delta \) and \( \lambda \): Weightage factors that shows to what extent the respective pressure group is not preferred. These are to be determined by local water authorities.

The maximum theoretical achievable PI is 1, which in reality is almost impossible to achieve and the PI will approach zero (0) for the very worst pressure conditions. The PI index is sensitive to the nominated weightage factor of each pressure category. Since the PI index is defined in the way to be flexible in pressures categories range and weightage factors, they can be modified based on priorities of each water authority. In this paper, based on the discussions with the respective water authority (Coliban Water), the adopted values of the weightage factors are: \( \alpha = 20, \beta = 5, \gamma = 3, \delta = 1, \) and \( \lambda = 2. \)

By adopting the mentioned weightage factors, the PI equation will be:

\[
PI = N_t \left( \frac{20 \left( N_{-\infty} \right) + 5 \left( N_{0} \right) + 3 \left( N_{5/20} \right) + 1 \left( N_{80/50} \right) + 2 \left( N_{80/\infty} \right)}{\left( N_{80/\infty} \right)} \right) + N_t
\]

**RESULTS AND DISCUSSION**

The PI has been calculated for the Castlemaine, Campbells Creek and Harcourt pressure zones for approved demands for planning target years of 2013, 2018, 2023 and 2038 in two following scenarios:

- Without water network upgrade.
- With approved water network upgrade.

Table 1 and Figure 3 show the overview of the modelled WDS in three case study townships along with the number of water connections per township. The number of connections per township has been received from the responsible water authority (Coliban Water).

| Table 1 | InfoWorks hydraulic model results overview for case study townships |
|---------|--------------------------|--------------------------|--------------------------|--------------------------|--------------------------|--------------------------|--------------------------|--------------------------|
|         | Calculated PI with no network upgrade | Calculated PI with network upgrade |
|         | Number of connections | 2013 | 2018 | 2023 | 2038 | 2013 | 2018 | 2023 | 2038 |
| Campbells Creek | 883 | 0.13 | 0.08 | 0.05 | 0.04 | 0.48 | 0.47 | 0.47 | 0.47 |
| Castlemaine | 3586 | 0.21 | 0.19 | 0.16 | 0.13 | 0.51 | 0.5 | 0.5 | 0.49 |
| Harcourt | 431 | 0.17 | 0.07 | 0.05 | 0.04 | 0.49 | 0.48 | 0.47 | 0.47 |
The PI has been calculated for the three case study townships to assess the impact of new demands on the water systems without applying any upgrades. As shown in the Campbells Creek water network, the PI drops from 0.13 to 0.04 in 2013 and 2038, respectively. The decrease in PI is occurring in the Harcourt system with a more intense rate from 0.17 to 0.04.

The same dropping pattern can be seen for the Castlemaine water system from 0.21 to 0.13 in the same period. However, the PI in 2038 without upgrading works in the Castlemaine system suggests that this water system is less sensitive to the new demands.

On the other hand, all three water systems show the notable increase in PI after applying the approved network upgrade works. The PI stays in the range of 0.47–0.51 after upgrade works which suggests the acceptable range of PI from the water authority’s point of view.

A comparison between PI figures for each town in studied planning years before and after network upgrade works, allows the PI variation or ΔPI calculation. This term is a useful figure to show the effectiveness of the planned upgrade works. This is also an indicator of the extent of the needed upgrade works in every planning year. Table 2 shows the variation in PI for each town in the target years before and after the network upgrade works.

Table 3 shows the associated cost of approved network upgrade works for case study townships per planning target year. This information has been received from Coliban Water which is the responsible water authority for asset planning in the study area. The approved upgrade works were pipe replacement, pipe duplication, new pressure reducing valve installation and network rezoning. The cost estimation was based on the database of Coliban Water from current upgrade works and the last five years of augmentation cost.

By dividing the associated cost of approved network upgrade works for case study townships by the townships’ number of connections, the cost per connection figure can

<table>
<thead>
<tr>
<th>PI variation - ΔPI</th>
<th>2013</th>
<th>2018</th>
<th>2023</th>
<th>2038</th>
</tr>
</thead>
<tbody>
<tr>
<td>Campbells Creek</td>
<td>0.35</td>
<td>0.39</td>
<td>0.42</td>
<td>0.45</td>
</tr>
<tr>
<td>Castlemaine</td>
<td>0.30</td>
<td>0.31</td>
<td>0.34</td>
<td>0.36</td>
</tr>
<tr>
<td>Harcourt</td>
<td>0.32</td>
<td>0.41</td>
<td>0.42</td>
<td>0.43</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Associated upgrade cost (AUD)</th>
<th>2013</th>
<th>2018</th>
<th>2023</th>
<th>2038</th>
</tr>
</thead>
<tbody>
<tr>
<td>Campbells Creek</td>
<td>845,000</td>
<td>849,500</td>
<td>871,000</td>
<td>891,000</td>
</tr>
<tr>
<td>Castlemaine</td>
<td>3,100,000</td>
<td>3,210,000</td>
<td>3,425,000</td>
<td>3,440,000</td>
</tr>
<tr>
<td>Harcourt</td>
<td>401,500</td>
<td>418,000</td>
<td>423,000</td>
<td>432,000</td>
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</tbody>
</table>
be calculated. As expected for each town, the cost per connection has an increasing trend between 2013 and 2038. Table 4 presents the system upgrade cost per connection.

In order to establish a relationship between improvement in PI and associated costs, values of \( \Delta PI \) (from Table 2) were plotted with their corresponding water system upgrade costs per connection (Table 4). Figure 4 shows the relationship between \( \Delta PI \) and the associated costs. While \( \Delta PI \) is in the range of 0.30–0.43, Figure 4 presents the budget required to achieve any intended improvement in the PI. The cost in the range of $864–1009 per connection is expected expenditure which will provide the mentioned improvement in the PI.

The relationship between \( \Delta PI \) and the upgrade cost per connection can be defined via two suggested equations:

1. Sigmoidal function:

\[
UC = \frac{a}{1 + b \cdot e^{-c \cdot \Delta PI}}
\]

where \( UC \) is upgrade cost per connection, \( \Delta PI \) is PI variation before and after network upgrade, and \( a, b \) and \( c \) are constants with the values of \( 9.91 \times 10^2 \), \( 6.96 \times 10^2 \) and \( 2.83 \times 10^1 \), respectively.

Figure 5(a) shows the regression of the suggested function with the observed data.

2. Polynomial function (degree 4):

\[
UC = a + b \cdot \Delta PI + c \cdot \Delta PI^2 + d \cdot \Delta PI^3 + e \cdot \Delta PI^4
\]

where \( a, b, c, d \) and \( e \) are co-efficients having values of \( -2.04 \times 10^{-3}, -3.15 \times 10^4, 2.95 \times 10^5, -8.17 \times 10^5 \) and \( 7.41 \times 10^5 \), respectively.

Figure 5(b) shows the regression of the suggested function with the observed data.

**CONCLUSIONS**

Water network upgrades and augmentations require significant budget allocation, which is one of the main concerns for water authorities in the development of water systems upgrades. So far, the undertaken studies in the field of water network planning predominantly have been focused on optimization of proposed upgrade works. In this research, a new methodology has been introduced to establish a relationship between ‘level of service’ and ‘upgradation cost’. A new

<table>
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<th>Table 4</th>
<th>The associated upgrade cost per connection for each planning year target</th>
</tr>
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<tbody>
<tr>
<td>Upgrade cost per connection (AUD)</td>
<td>2013</td>
</tr>
<tr>
<td>Campbells Creek</td>
<td>957</td>
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<tr>
<td>Castlemaine</td>
<td>864</td>
</tr>
<tr>
<td>Harcourt</td>
<td>932</td>
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</tbody>
</table>

Figure 4 | The associated upgrade cost per connection for respective PI improvement.
index has been introduced to evaluate the level of service of WDS from a pressures point of view. The index has been named as ‘Pressure Index’ (PI) since it targets the evaluation of WDS level of service based on occurring water pressures in the system for current and future populations.

The study was conducted for three Australian townships in Castlemaine WDS area as a case study. An InfoWorks WS hydraulic model was developed for four planning target years of 2013, 2018, 2023 and 2038. The first three planning target years were adopted as the staging target and the year 2038 was chosen as the ultimate planning target year. The information about the water demands forecast, approved upgrade strategy and the cost estimation were received from Coliban Water which is the responsible water authority for asset planning in the study area. The study has shown that the PI factor drops due to increasing demand in the case study townships if no upgrades are undertaken. By applying the approved upgrade works, the PI factor rises to the range of 0.47–0.51. This range seems to be the required PI for an acceptable level of service provided by approved upgrade strategy. In other words, this range shows the minimum needed PI and, therefore, will indicate the extent of upgrade and augmentation works to cater for future population in the serviced area.

It is found that in the case study townships, the upgrade budget in the range of $959–1009 per water connection is needed for the ultimate planning target year (2038) to achieve

Figure 5 | (a) Suggested sigmoidal function for relationship between ΔPI and upgrade cost per connection. (b) Suggested polynomial function for relationship between ΔPI and upgrade cost per connection.
the PI in the minimum acceptable range of 0.47–0.49. The research identified that in the case study townships there is a meaningful trend between PI variation (ΔPI) and the associated water network upgrade cost. Two mathematical functions are introduced to define the relationship between PI variation (ΔPI) and the associated water network upgrade cost in the case study townships. It should be noted that the three studied townships do not have many high rise buildings or high density populated areas. Further studies on other kinds of water systems with more complex system controls, different acceptable level of services and various types of connections are needed to achieve more comprehensive results with a generalised trend. The type of regression that has been used in this paper can also be modified and improved by utilisation of different mathematical formulations.

Therefore, the methodology introduced in this paper can be elaborated further by other researchers for other populated areas around the world considering the local ‘level of service’ regulations for minimum and maximum pressures.

**REFERENCES**


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