Holistic planning methodology for long-term design and capacity expansion of water networks
T. T. Tanyimboh and P. Kalungi

ABSTRACT

The application of the analytic hierarchy process (AHP) to help select the best option for the long-term design and capacity expansion of a water distribution network is described and applied to a sample network. The main criteria used are: reliability-based network performance; present value of construction, upgrading, failure and repair costs; and social and environmental issues. The AHP has been applied elsewhere on various problems, but not on the long-term upgrading of water distribution networks as proposed in this paper. The pipes are sized to carry maximum entropy flows using linear programming while the best upgrading sequence is identified using dynamic programming. The example demonstrates the effectiveness of the AHP as a systematic tool for assessing pareto-optimal designs based on the trade-offs between multiple criteria. The results demonstrate that the cheapest option is not necessarily the best when other factors e.g. performance and socio-environmental concerns are considered in an explicit way.

Key words | analytic hierarchy process, informational entropy, linear programming, multicriteria optimization, reliability, water distribution networks

INTRODUCTION

The long-term upgrading of a water distribution system (WDS) should address economic, social, environmental, health, hydraulic and other technical issues. This is a multi-objective problem with a high level of complexity. This paper presents a model for a staged design and upgrading strategy. The significance of the proposed method lies in its ability explicitly to consider the deterioration over time of both the structural integrity and hydraulic capacity of every pipe, and to allow for the direct and indirect failure costs. The model identifies pipes for upgrading or rehabilitation based on the options of paralleling and replacement. The overall design horizon is subdivided into two phases whose durations are automatically optimised by the model using dynamic programming. The second phase allows for paralleling or replacement of some pipe sections where appropriate. Evidence in the literature suggests that designs with maximum entropy flows are generally more reliable than conventional designs (Tanyimboh & Templeman 2000). Accordingly, the pipes are designed to carry maximum entropy flows (Yassin-Kassab et al. 1999), which also enables the pipe-size optimization to be performed using linear programming. Relining an existing pipe with a smooth liner, after scouring away encrusted material, was not included as one of the options for upgrading the network mainly because it is limited in diameter by the existing pipe. In any case, relining is generally not much cheaper than paralleling or replacement, especially for pipe sizes less than 250 mm in diameter (Walski et al. 2001). Also, partly because of the complexity of the problem, energy costs have not as yet been included in the present model.

The overall objective is to minimise the total cost associated with the pipes including installation, breakage, repairs, and upgrading. The method used for assessing the trade-offs between the various optimal designs in this research is a ranked paired-comparison technique called the analytic hierarchy process (Saaty 1980). The analytical
The design methodology and upgrading options

As mentioned earlier, the overall design horizon is subdivided into two phases whose durations are automatically optimised by the model using dynamic programming. The segmental-pipe linear programming approach (Alperovits & Shamir 1977) was used for optimizing the pipe diameters. The decision variables are the lengths of the pipe segments.

Design constraints

The design constraints are the governing equations for flow in WDSs, i.e. pipe head loss; nodal flow continuity; energy conservation; maximum and minimum demand node pressures; and the imposed maximum and minimum pipe flow velocities. The pipe roughness increases over time at a rate that varies according to the pipe type, water quality and operation and maintenance practices. To model the effect of aging on the carrying capacity of pipes, the equation of Sharp & Walski (1988) was used, i.e.

\[ C_{ijm}(t) = 18.0 - 37.2 \log \left( \frac{e_{ijm} + a_{ijm}(age_{ijm})}{D_{ijm}} \right) \quad \forall ijm \]  

(1)

where, for segment \( m \) of link \( ij \), \( C_{ijm}(t) \) = Hazen-Williams coefficient in year \( t \); \( e_{ijm} \) = roughness (mm) at time of installation; \( a_{ijm} \) = roughness growth rate (mm/year); \( age_{ijm} \) = number of years from installation; \( D_{ijm} \) = segment diameter (mm). The demand is the value at the end of the relevant design phase, which was obtained by projecting the nodal base demand from the first year of the entire design horizon to the end of the relevant design period. The annual rate of increase of the base demand was assumed to follow demographic patterns closely and approximated by the population growth rate.

Details of the various costs involved

The cost \( C_r(s, r, d) \) of adding capacity \( r \), in each design phase \( \tau \) is a function of the added capacity, as well as the existing capacity \( s \), at the beginning of the design phase. It was assumed that construction costs are incurred at the beginning of each period. Capital is to be raised by borrowing at an annually compounded interest rate of \( b\% \). All borrowed capital has to be paid back at the end of the design horizon. Thus, the overall cost objective function represents the present value of the total end debt as follows:

\[ \text{Cost} = \sum_{\tau=1}^{2} \beta_{\tau} C_r(s_\tau, r_\tau)(1 + b)^{(d - \tau)} \]  

(2)

where:

\[ C_r(s_\tau, r_\tau) = f_1 + f_2 + f_3 \]  

(3)

\( f_1 \) represents pipeline costs, which include installation, paralleling, replacement and repair costs; \( f_2 \) is the cost of setting up the construction plant and machinery at the beginning of each phase; \( f_3 \) represents costs that vary with the magnitude of the installed capacity. Further characterizations of \( f_1 \), \( f_2 \) and \( f_3 \) follow below. The term \( (1 + b)^{(d - \tau)} \) is the compound factor; \( d \) = design horizon (in years); \( v = 0 \) when \( \tau = 1 \); and \( v = T1, \ldots, T2 \) when \( \tau = 2 \); \( T1 \) is the lower limit for the end of Phase I and \( T2 \) is the upper limit for the end of Phase I. Equation (2) represents the costs for a two-phase design and upgrading sequence. \( \beta_\tau \) is the product of a discount factor, \( (1 + r)^{-\tau} \), and a price increase factor, \( (1 + c)^\tau \). This means that the discount rate is assumed to be \( r\% \) per annum, in each period \( \tau \), and there is a general increase in construction costs at a rate of \( c\% \) per annum. It was assumed that the values of \( c \) and \( r \) are equal, thus \( \beta_\tau = 1 \). This assumption was made because this study is an economic analysis aimed at comparing alternatives based on the present value of their costs. This, however, may not apply to a financial analysis, which aims at assessing the profitability of a project with an emphasis on...
where \( f1a \) and \( f1b \) represent costs of new and parallel pipelines, respectively, and include supply, installation and the future failure costs. The costs \( f1a \) and \( f1b \) are identical because each one concerns newly supplied pipes installed in freshly excavated trenches. All pipes used for this study are made of polyvinyl chloride (PVC). Thus

\[
f1 = f1a + f1b + f1c
\]  

(4)

in which \( D_{ijm} \) and \( l_{ijm} \) = diameter and length respectively (in meters) of pipe segment \( m \) of link \( ij \); \( N_{ij} \) = number of segments in link \( ij \); \( \gamma_p \) and \( c_p \) are constants. The first term in the outer brackets represents the cost for the supply and installation of new and parallel pipes. \( REP_{ijm} \) = failure costs consisting of the present value of direct and indirect failure costs for pipes older than 5 years. New pipes were assumed to have no repair costs during their first 5 years as they are usually under warranty. The failure costs are given by Dandy & Engelhardt (2001) as

\[
REP_{ijm} = \sum_{t = ts}^{tr} \frac{J(t)_{ijm} \cdot CB_{ijm} \cdot FCF(LU_{ij}) \cdot l_{ijm}}{(1 + r)^{t-ts+1}} \quad \forall ijm
\]  

(6)

where \( r \) = discount rate; \( ts \) = first year of a given design phase; \( tr \) = last year of a design phase; \( t_b \) = time from which a pipe starts to incur repair costs (sixth year). To obtain the best rehabilitation strategy for an existing network in the first design phase, \( ts = tr \); i.e. the time for the first phase is set equal to the design life of the existing network. \( FCF(LU_{ij}) \) is the failure cost factor for land use, \( LU_{ij} \), for link \( ij \). The failure cost factors cater for indirect costs caused by pipe failures, e.g. disruption to traffic and damage incurred by third parties. \( CB_{ijm} \) = repair cost per break; \( J(t)_{ijm} \) = break rate (breaks/km/year) in year \( t \). The repair costs per break (Dandy & Engelhardt 2001) were adjusted using construction indices for the general annual increase in construction costs to obtain figures for 2002. Thus

\[
CB_{ijm} = \gamma_{fr}(D_{ijm} \cdot 1000)^\Phi; \forall ijm
\]  

(7)

where \( \gamma_{fr} \) and \( \Phi \) are coefficients specified by the user. Following Dandy & Engelhardt (2001), it was assumed that the break rate for PVC pipes is given by

\[
J(t)_{ijm} = 0.001974 \cdot \exp(-0.00974 \cdot D_{ijm}) \cdot \gamma_{fl}^{1088} \\
\forall ijm
\]  

(8)

where \( age_{ijm} \) is the number of years from installation.

The cost \( f1c \) of replacing deteriorated pipes covers the pipe replacement costs together with the repair costs associated with the new pipes. The pipe replacement costs were assumed to be about 5% higher than parallel pipe costs (Directorate of Water Development 1999).

\[
f1c = \sum_{i \in I} \sum_{m=1}^{N_i} \gamma_{cr} \cdot (c_r \cdot D_{ijm}) \cdot l_{ijm} + REP_{ijm} \\
\forall ijm
\]  

(9)

where the diameter \( D_{ijm} \) and length \( l_{ijm} \) are in metres; the first term in the outer brackets represents the pipe replacement costs; \( \gamma_r \) and \( c_r \) are coefficients specified by the user. \( REP_{ijm} \) (Equation 6) represents the repair costs associated with these new pipes. The decision variable is \( l_{ijm} \).

The cost \( f2 \) of setting up the construction plant at the beginning of each phase also includes the mobilisation and setting up of all the necessary facilities for the contractor and his workforce. The costs \( f3 \) that may vary with the magnitude of the installed capacity in a particular phase, were included on the grounds that the total volume of water released from a reservoir into the system has a proportion of the costs attributed to various ancillary factors including: treatment and transmission of water; capital costs associated with an increase in water supply (e.g. new wells and extra sewage treatment capacity); operation and maintenance costs, etc. (Directorate of Water Development 1999). A generalised relationship was assumed to represent all these costs:

\[
f3 = VC \cdot Q_{inst}^{VE} \\
\]  

(10)

where \( Q_{inst} \) is the installed capacity in a particular design phase in l/s, \( VC \) and \( VE \) are coefficients whose values depend on the above mentioned factors and are specified by the user.
The analytic hierarchy process

In the AHP, goals, criteria and alternatives are arranged in a hierarchical structure. The top level consists of the ultimate goal; the second level the criteria by which the alternatives are evaluated; and the third level the sub-criteria for each criterion, and so on. The lowest level consists of the design or policy options. Herein, the first level stipulates that the overall goal is to select the best design option i.e. one that minimizes the cost and environmental damage and maximises network performance and social benefits. The second level has three main criteria: (a) network performance; (b) economic value; and (c) social and environmental issues. The third level has the sub-criteria and the fourth level consists of the design options (Figure 1).

The economic value is expressed in terms of the present value of the costs for each of the design options. The network performance is defined in terms of its hydraulic reliability and the failure tolerance. The hydraulic reliability is considered the network’s ability to satisfy customer demands at adequate pressure under normal and abnormal operating conditions. The failure tolerance is taken as the proportion of the total network demand that is satisfied on average during periods in which some components are unavailable. Taken together, the hydraulic reliability and redundancy address the levels of service concerns including the underlying pipe breakage rates and mechanical reliability of other components.

The social and environmental issues addressed include acceptability, abstraction and health concerns. Acceptability is the project beneficiaries’ attitude towards a particular upgrading strategy and whether they would tolerate it. For example, a pricing strategy which involves increasing the price of water to reduce consumption and so delay the capacity expansion may not be favourable to the consumers or acceptable to the regulatory agencies. Abstraction refers to the potential for adverse environmental and ecological impacts due to an increase in water consumption. A water tariff increase which reduces consumption and thus abstraction, might be more environmentally friendly. Health issues address the risk that a particular option might lead to the health of consumers being compromised through the use of alternative cheap, unsafe water sources or rationing of water usage to a level below the minimum required for essential hygiene.

After the creation of a hierarchy to represent the problem, a matrix of pair-wise comparisons is used to rate the criteria, using a nine-point scale ([Saaty 1980](#)) which expresses the relative importance or superiority of one criterion compared to another. A priority vector, obtained as follows, is used to rate the criteria. The elements of the comparisons matrix are normalized by dividing each element by the corresponding column total. A priority

![Figure 1](https://iwaponline.com/ws/article-pdf/8/4/481/418984/481.pdf)
vector is then obtained as the average of the elements of each row of the normalised comparisons matrix.

Using the available data, each option is rated with respect to each subcriterion by performing pair-wise comparisons between the options. These comparisons show how much more one project fulfils a given subcriterion compared to another project. Project priority vectors are then obtained from the comparisons matrices as explained above. The next step involves the generation of a project priority vector for each primary criterion. This is done by assembling the subcriteria project priority vectors for each parent (i.e. primary) criterion in a matrix called the project priority matrix. The project priority vector, for each parent criterion, is obtained by multiplying its project priority matrix by the priority vector for its subcriteria. Finally, the overall ratings of the projects are obtained in an analogous fashion by multiplying the matrix of the project priority vectors for the primary criteria by the priority vector for the primary criteria (Saaty 1980).

**RESULTS AND DISCUSSION**

Figure 2 shows the three design options for a simple network, which permits a more complete presentation of the results. Results for a real network can be found in Kalungi (2003). The design horizon is \( d = 20 \) years. A Fortran 95 program was developed and executed on a PC (128 MB RAM; 1.2 MHz microprocessor speed). The nodal demands in the first year for Nodes 2, 3, 4 and 5 are 3, 4, 12 and 12 l/s. 25% of a fire demand of 20 l/s at Node 2 (Twort et al. 2000) along with the peak hour demands with a peak hour factor of 2.0 (Directorate of Water Development 1999) were used. The base demands were assumed to increase at an annual rate of 4%. The design demands for Option 1 are smaller due to a one-off tariff increase of 33.33% two years to the end of each design phase. The price elasticity of demand was taken as -0.3 (Dandy et al. 1985). The reliability and failure tolerance values were calculated using head-dependent modelling. The details of the reliability and failure tolerance calculations, and the approach used for the head-dependent modelling are in Kalungi (2003) and Kalungi & Tanyimboh (2003).

All links are 1,000 m long; pipes are made of PVC; water level at the source = 70 m; all demand nodes have elevations of 0 m and desired service heads of 15 m; Hazen-Williams coefficient = 130 for all new pipes; \( b = 8\% \); \( v \) for Phase II varies from 7 to 13 years i.e. \( T1 = 7 \) and \( T2 = 13 \); all costs

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**Figure 2** | Simple demonstration network. Option 1: maximum entropy flows with tariff increase, Option 2: maximum entropy flows without tariff increase, Option 3: shortest path flows (Orth 1986) without tariff increase.

**Table 1** | Hydraulic performance and design optimization results

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Option 1</th>
<th>Option 2</th>
<th>Option 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Phase I design variables</td>
<td>20</td>
<td>23</td>
<td>20</td>
</tr>
<tr>
<td>Phase II design variables</td>
<td>46</td>
<td>48</td>
<td>40</td>
</tr>
<tr>
<td>CPU time (seconds)</td>
<td>0.391</td>
<td>0.488</td>
<td>0.289</td>
</tr>
<tr>
<td>Phase I duration (years)</td>
<td>9</td>
<td>14</td>
<td>11</td>
</tr>
<tr>
<td>Phase II duration (years)</td>
<td>11</td>
<td>6</td>
<td>9</td>
</tr>
<tr>
<td>Phase I cost ($)</td>
<td>3,222,733</td>
<td>3,566,437</td>
<td>2,914,191</td>
</tr>
<tr>
<td>Phase II cost ($)</td>
<td>588,118</td>
<td>648,366</td>
<td>615,592</td>
</tr>
<tr>
<td>Total cost ($)</td>
<td>3,810,851</td>
<td>4,214,803</td>
<td>3,529,783</td>
</tr>
<tr>
<td>Failure tolerance</td>
<td>0.850584</td>
<td>0.934716</td>
<td>0.647552</td>
</tr>
<tr>
<td>Reliability</td>
<td>0.999553</td>
<td>0.999832</td>
<td>0.999484</td>
</tr>
</tbody>
</table>
are in US dollars; \( FCF(LU)_{ij} = 4 \) for all pipes; \( r = 8\% \);
\( e_{0im} = 0.0021 \) mm; \( ai_{jm} = 0.025 \) (mm/year);
\( g_p = 32.093 \); \( cp = cr = 3.7 \);
\( g_{br} = 33.928 \); \( F = 0.6067 \); minimum velocity = 0.5 m/s; maximum velocity = 3 m/s; maximum hydraulic gradient = 50 m/km (Twort et al. 2000); setting-up costs at start of each phase = $100,000; \( VC = 130 \); and \( VE = 1.6 \).

The results of the design and upgrading model are summarized in Tables 1 and 2. More detailed results, Options 1 and 3 included, can be found in Kalungi (2003). Table 3 shows the priority vectors for the primary and secondary criteria. Table 4 shows the project priority vectors and the overall priority vector. Option 1 is cheaper than Option 2 because Option 1 has a smaller capacity due to its two tariff increases which reduce the overall water consumption as shown in Figure 2 and delay the WDS capacity expansion. The indirect failure costs are sensitive to the timing and magnitude of the upgrading. This means that due attention is required when specifying indirect failure costs through the failure cost factors. Table 2 shows that paralleling tends to be preferable to replacement in Phase II (Kalungi 2005). Paralleling increases the capacity of the system, which cannot be achieved by relining. Thus the results support the decision to consider only paralleling and pipe size upgrading as the rehabilitation options in this formulation. The AHP rating of Options 1, 2 and 3 is 0.314, 0.375 and 0.310, respectively, as shown in Table 4. This means that Option 2 is the best and Option 3 is the least preferred. Since the AHP is a tool for assisting in the process of decision-making, one could perhaps argue that Options 1 and 3 are not significantly different and are therefore equally less suitable than Option 2. Table 1 shows that the present values of the project costs for Options 1, 2 and 3 are $3,810,851; $4,214,803; and $3,529,783 respectively. It is clear that the cheapest design, Option 3, also has a much smaller value of failure tolerance. Whereas Option 3 is the most favourable in terms of costs, it is the least favourable in terms of performance. The AHP is very helpful in handling such instances of conflicting decision factors.

From Table 1, the reliability values of the Options 1, 2 and 3 are 0.999553, 0.999832 and 0.999484, respectively. These values are very close to each other due to the fact that the individual component reliabilities of the networks are high. These results would probably not be enough for one to distinguish between the designs using this criterion alone.
On the other hand, the values of failure tolerance for Options 1, 2 and 3 are 0.850584, 0.934716 and 0.647552, respectively. The failure tolerance for Option 3 is the lowest and significantly different from that of Options 1 and 2, primarily because the layout for Option 3 is partially dendritic. For the sample network, failure tolerance clearly distinguishes between the designs and stresses the point that reliability and failure tolerance should be used together as performance assessment parameters.

Based on their experience, technical knowledge and knowledge of the project at hand, different analysts (i.e. decision makers) will in general obtain different criteria and project priority vectors. Where several analysts are involved, a compromise can be arrived at by calculating the weighted average of their respective overall project priority vectors. Their experience, technical knowledge and knowledge of the project at hand can be assessed using the AHP to yield a priority vector for the analysts. Assembling the analysts’ overall project priority vectors in a matrix and multiplying the matrix by the priority vector for the analysts results in the final overall priority vector for the group as a whole. Tables 3 and 4 show the results for one analyst (Decision Maker 1) whose overall project priority vector is (0.314, 0.375, 0.310). The overall priority vectors for Decision Makers 2 and 3 are (0.297, 0.356, 0.347) and (0.521, 0.383, 0.295), respectively. Assembling these three priority vectors in turn for the decision makers gives the results in Table 5. Table 5 shows that the best design and upgrading strategy is Option 2 in all four cases, thus suggesting the results of the AHP are stable.

**CONCLUSIONS**

The long term design and upgrading model presented explicitly considers deterioration over time of both the structural integrity and hydraulic capacity of every pipe and allows for the direct and indirect failure costs. It simultaneously considers the upgrading options of paralleling and replacement of pipes and identifies the pipes to be upgraded along with their timings. The use of maximum entropy flows simplifies the design problem without sacrificing reliability and redundancy, and would appear not to lead to excessive costs. The results show that in general paralleling is preferred to replacement probably because it is cheaper and the rate of deterioration of the pipes is quite low. For the example considered, the most favourable design in terms of costs turned out to be the least favourable in terms of performance. Thus the cheapest option is not necessarily the best because other factors can weigh it down. Therefore, it is important to consider all the relevant qualitative and quantitative factors explicitly in the decision-making process. The AHP proved to be very useful in this regard.
effective at handling such instances of conflicting decision factors and identifying the best solution in terms of cost, performance and the wider social, health and environmental issues. Sensitivity analysis confirmed that the final decision regarding the best design option is very stable and not overly sensitive to changes in the weights carried by the various decision makers. The overall CPU time of approximately 0.5 seconds on a PC is quite low considering that it also covers different linear programming designs for the various Phase I and II periods, together with dynamic programming to determine the best upgrading sequence. The CPU time for larger networks with about 25 links is in the region of about 5 seconds (Kalungi 2003).

REFERENCES


