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An Optimization Model for Drinking Water Pipeline Network Considering Pipe Deterioration

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ABSTRACT

Recently, the management of drinking water pipelines must face challenges. Because these pipes are mostly located underground, they are vulnerable to damage overtime. This damage causes problems such as rupture and leakage in the pipe. This paper proposes to optimally select the appropriate pipe diameter during pipeline replacement planning for drinking water pipelines using an integer programming model. First, the problem was formulated to minimize replacement costs (economic perspective) by considering hydraulic constraints namely stability of flow velocity for each pipe and water pressure at each node. We solved the model using an improved direct search approach.

Keywords: Optimization, Water pipeline network; Replacement planning; Integer programming.

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INTRODUCTION

Management of water pipelines may raise explicit issues. A water pipe network refers to a category of resources recognized as linear assets, such as roads, rail lines, electricity lines, gas and oil pipelines, or telecommunications networks. After a while, the pipelines deteriorate in underground water distribution systems. This worsening of the water pipelines contributes to malfunctions such as leakage and breakage, consequently, causes a lack of fresh water, emergency and unplanned maintenance, disruption of water supply, damage to property or losses. Many of these impacts appear to be interlinked and may be compounded in extremely costly cases.

Optimizing the pipe network involves calculating a new pipe network and rehabilitating the current network. There are two hydraulic regulations for the water distribution system: the demand for water and the pressure head in the supply areas. There have been three methods of optimization, the lowest cost model, the highest profit design and the cost-benefit trading system, Wu et al. [1]:

- The least cost optimization is the quest for the ideal alternative by minimizing costs whereas meeting development constraints. However, the lowest price optimization generates the minimum size of the pipe to reduce the capacity and reliability of the supply.
- Optimization of the highest point profit design maximizes the return on the funds consumed by seeking the most profitable alternative within an accessible budget while remain complying with hydraulic limitations.
- Optimizing the cost-benefit balance using a multi-objective design model is accomplished to minimize costs and maximize benefits while meeting constraints.

Typically, almost all of the water distribution network design work was aimed at creating optimization processes to address the lowest price pipe size issue.

Throughout water distribution schemes, various optimization techniques are applied. There are stochastic optimization methods such as genetic algorithms, simulation annealing and deterministic types for optimization such as linear programming, non-linear. Djebedjian et al. [2] summarizes how these approaches are used in water distribution networks. Genetic Algorithms (GAs) have been widely used to optimize hydraulic requirements for water distribution systems. GAs greatest assets are that they use a population of changing alternatives and recognize a number of options that can be chosen by the decision-maker instead of a single optimum. The primary drawback is the increased computing strength.

The literature review indicates that optimization is common in simple water distribution systems. Large-scale development of water networks has certain significant characteristics, such as population overgrowth and topography. Some articles focus on information on specific management planning research or on proper water distribution systems.

Rayan et al. [3] used a sequential, unrestricted minimization method to optimize the El-Mostakbal City network, as well as the existing Ismailia City, Egypt distribution network. The expansion network consists of 31 nodes and 43 tubes. Letha and Sheeja [4] used the basic cost-and-loss ratio technique and applied it to a real field issue: the area in Thiruvananthapuram City, Kerala State.

In order to demonstrate the practical implementation of the GA, Shau et al. [5] chose the Ruey-Fang water supply system in Taipei County. There are 26 pipelines, 20 nodes, and 2 water intake sites. Water treatment plant is one of intake, while Kung-Liao system support is the other.

Wu et al. [1] has researched a water distribution system that supplies approximately 18.3 million gallons of water per day for peak demand. The hydraulic network model includes 2018 tubes, 1371 nodes, 3 valves, 1 reservoir, 9 high-power units and 20 wells. Optimization processes have been applied in four circumstances to refine the alternatives for change: to achieve complete demand development, to evaluate sustainable demand growth, to increase water production and to prioritize capital improvement phases.

ANALYSIS RELIABILITY

Reliability analysis acts an important role in enhancing the capacity of the water pipe network. The performance of the system is likely to meet its primary purpose under specified conditions for a specified period of time [6].

Prior to 2001, Kleiner and Rajani [7] carried out a thorough assessment of the statistical models for structural deterioration of water systems. By observing the details of historical performance, they tried to calculate the structural deterioration of the water mains. Statistical methods have been classified into deterministic and probabilistic models. The analysis presented summaries of the different models, including their predominant calculations, as well as feedback, correlation and description of the types of data needed for implementation. Several efforts have been made in recent years to achieve stronger prediction results for water pipe errors.

Existing models frequently consider all pipes in the pipe system when evaluating the efficiency of the water pipes. Water pipes, however, are usually linear properties; they have no definite physical boundaries and generally cover long distances that may be classified into sections [8]. Each section has the same role, but separate burdens and environmental circumstances may apply. The accuracy of all other sections cannot be affected by the failure of a single segment of the pipe. As a result, a water pipe should be seen as a number of segments. However, most current models do not recognize the personal contribution of different pipe sections to the reliability of the system.

There are two questions to answer: how many groups should be divided, and what conditions should be used to create a group? The number of subgroups should be balanced by two elements: (1) homogeneity in each subgroup, and (2) sufficient information on the risk calculation error. The more divided the groups, the more homogeneous the characteristics are within each community, but there are fewer observations remaining for statistical analysis in each subgroup.

The literature offers a range of methods that rely on specific features to divide water pipe data into populations. Some classify pipes according to specialized engineering expertise [9]. This kind of strategy has the advantage that pairing focuses on practical knowledge of the characteristics of the pipe and its failure methods. For example, different substances have different physical characteristics that can contribute to different types of error and failure levels. However, only materials and ages are taken into account in these strategies. A *ANOVA* method was developed to analyze error data [10]. It groups the break data and sets the break frequency patterns for each group. However, in order to validate the results of the grouping, the criteria of the grouping must be chosen, first on the basis of the previous understanding, before ANOVA. In particular, it is necessary to investigate the prior understanding of the grouping criteria. In addition, it is assumed that an exponential increase over time meets the rate of breakage, which in some cases is not consistent with the facts.

OPTIMIZATION MODEL FORMULATION

The objective of the optimization of the water distribution network is to identify suitable pipe diameters for the specific design and demand needs of the network. Optimal tube dimensions and limitations (e.g. hydraulics and structural requirements) will be chosen for mass and energy conservation in the initial network.

The equation (1) is the objective function for the total network cost notated by C_T:

$$C_T = \sum_{i=1}^{i=N} c_i(D_i)L_i \tag{1}$$

where D_i per unit length, L_i is the length of pipe i, c_i (D_i) the cost of pipe i with diameter and N is the total number of pipes.

Decreased prices for the network have been defined by the conservation of mass and conservation of energy. Mass storage means, with the exception of reservoirs and tanks as storage nodes, that the release into each node is equivalent to that left by the node. This constraint can be written for the total number of M nodes on the network:

$$\sum_{j=1}^{M} Q_j = 0 \tag{2}$$

where Q_j is the discharge of the j node (including the sign).

Energy conservation states that if a pump E_p exists, the total head loss around each loop must be zero or the energy supplied by:

$$\sum h_f = E_p \tag{3}$$

where h_f is a loss of head in a pipe due to friction. It requires that the head loss of any pipe, which depends on the size, width and hydraulic characteristics of the pipe, is equivalent to the disparity between the nodal heads.

Various forms of head loss equations have been produced for functional pipe flow calculations. The Hazen Williams equation explains the hf head loss in the pipe in this study:

$$h_f = \frac{10.6744}{C_i^{1.852}} \frac{L_i Q_i^{1.852}}{D_i^{4.8704}} \tag{4}$$

where C_i is the Hazen-Williams coefficient, Q_i is the pipe flow (m^3/s), L_i is pipe Length (m) and D_i is pipe diameter (m).

Formula total cost of replacement and discretized

The summary of replacement costs and losses of one of the pipes replaced by $\tau*$ during a planning period T is shown as:

$$C_{tot,\tau^*} = C_{repl} + C_{fail} \cdot Nseg \cdot \int_0^{\tau^*} f_{crp}(\tau) d\tau + C_{fail} \cdot Nseg \cdot \int_{\tau^*}^{T} f_{crp}(\tau - \tau^*) d\tau \tag{5}$$

where Crepl is the replacement cost Equation (5) contains three parts:

(1) C_{repl} is the cost of replacement during the T (which may or may not occur);

(2) $C_{fail} \cdot Nseg \cdot \int_0^{\tau^*} f_{crp}(\tau) d\tau$ indicates the failure cost prior to the replacement activity at age τ^*;

(3) $C_{fail} \cdot Nseg \cdot \int_{\tau^*}^{T} f_{crp}(\tau - \tau^*) d\tau$ is the cost of failure after the replacement activity, where reliability meets the declining pattern of" as good as new" age from the beginning of τ^*.

Commonly, the repaired time is reported on the date or year of operation. Then (5) is changed in a discretised such that total cost of plan year T is given as follows:

$$C_{repl} + C_{fail} \cdot Nseg \cdot \sum_{\tau=1}^{\tau^*} f_{crp}(\tau) + C_{fail} \cdot Nseg \cdot \sum_{1}^{T-\tau^*} f_{crp}(\tau - \tau^*) \tag{6}$$

where τ is a discretised age in year, τm = 1, 2, ..., T.

Cost formulas based on planning year t

Typically, the substitution system is linked to the calendar year instead of the age. Accordingly, the age-specific total cost of τ should be converted from the planning year to the calendar year-specific total cost of t. Let *cur D* be the actual date of installation in year and let $instD_i$ be the date of installation of each i pipe in year.

Throughout the preparation horizon T, the cost of repairing the pipe i for its calendar year τ^* (τ^* = 1,2,...,τ) is as follows:

$$\begin{aligned} C_{fail,i,t^*} = &\sum_{t=1}^{t^*} \left[f_{crp,i}(t + currD - instD_i) \cdot C_{fail,i} \cdot Nseg_i \right] \\ &+ \sum_{t=1}^{T-t^*} \left[f_{crp,i}(t) \cdot C_{fail,i} \cdot Nseg_i \right], \quad (t = 1, 2, \ldots, T) \end{aligned} \tag{7}$$

where $Nseg_i$ is number of segments of pipe i.

Assuming that only one alternate procedure can be carried out throughout the planning horizon T, the total cost of elimination of pipe i during the planning horizon T during its calendar year t^* is as follows:

$$C_{repl,i,t^*} = C_{repl,i} \tag{8}$$

The total cost of installation pipe i of planning T is therefore estimated for the t^* calendar year

$$C_{i,t^*} = C_{repl,i,t^*} + C_{fail,i,t^*}$$

The objective must be minimized under the constraints. These limitations are the development and hydraulic constraints. Structural constraints (maximum and minimum pipe diameter thresholds) and hydraulic constraints were defined as:

:

$$D_{min} \le Di \le D_{max} \qquad i = 1, \ldots, N \tag{9}$$

$$H_{j,min} \le H_j \le H_{j,max} \qquad j = 1, \ldots, M \tag{10}$$

where H_j is the pressure head at node j, $H_{j,max}$ and $H_{j,min}$ are the maximum and minimum allowable pressure heads at node j.

Due to the fact that the availability of pipes in markets is in certain diameter, then the problem becomes a nonlinear integer programming model.

1. THE BASIC APPROACH

It is worth considering the core strategy of the linear scenario system, i.e., Integer Linear Programming (*ILP*) problem until we pass on problems with *INLP.*

The equation (11-14) be consideration of a *MILP* problem

$$\text{Minimize } P = c^T x \tag{11}$$

$$\text{Subject to } Ax \leq b \tag{12}$$
$$x \geq 0 \tag{13}$$
$$x_j \text{ integer for some } j \in J \tag{14}$$

If vector $(x_B)_k$, be component of optimal basic and feasible, then *MILP* solved is a continuous can be expressed as the following:

$$(x_B)_k = \beta_k - \alpha_{k1}(xN)1 - \cdots - \alpha_{kj^*}(x_N)_j - \cdots - \alpha_{k\ n-m}(x_N)_{n-m} \tag{15}$$

The table 1 of the simplex technique at the end. If $_k$ is not an integer and $(x_B)_k$ is an integer, $_k$ can be partitioned into:

$$\beta_k = [\beta_k] + f_k, \quad 0 \leq f_k \leq 1 \tag{16}$$

Presume we want to multiply $(x_B)_k$ to its nearest integer, ([ ] + 1). On the basic of the theory of suboptimal solutions, we can increase a certain parameter i.e. $(x_N)_{j^*}$, above its zero limit, given that $_{kj^*}$, is a negative element of the $_{j^*}$ vector. That Δ_{j^*} is a non-variable $(x_N)_{j^*}$ movement number, so that the numerical scalar value $(x_B)_k$ is an integer. With referring to Eqn. (15), Δ_{j^*} can be expressed as the following:

$$\Delta_{f^*} = \frac{1 - f_k}{-\alpha_{kj^*}} \tag{17}$$

while the remaining non-basic is zero. After (16) replacing $(x_N)_{j^*}$ into in (17) and taking into account the $(x_B)_k$ splitting in (16), we can see that:

$$(x_B)_k = [\ \ ] + 1$$

Thus, $(x_B)_k$ is now an integer.

The non-basic variable is now apparent and plays a significant role for integerizing of related basic variable. Therefore, the following finding are important to verify that a non-integer variable must be used to function is the integerization system.

Theorem 1. *Let (*11) – (14) *has an optimal solution for the MILP problem, then some of the non-basic variables.* $(x_N)_j$, *j* = 1, 2, n, *must be non-integer variables.*

Proof:

Solving problem as a continuous of slack variables (which are non-integer, except in the case of equality constraint). If we assume that the vector of basic variables x_B consists of all the slack variables, then all integer variables would be in the nonbasic vector x_N and therefore integer valued.

Derivation of method

It is clear that the other components, $(x_B)_{i\neq k}$, of vector x_B will also be affected as the numerical value of the scalar $(x_N)_{j^*}$ increases to Δ_{j^*}. Consequently, if some element of vector $_{j^*}$, i.e., $_{j^*}$ for $i \neq k$, are positive, then the corresponding element of x_B will decrease, and eventually may pass through zero. However, any component of vector x must not go below zero due to the non-negativity restriction. Therefore, a formula, called the minimum ratio test is needed in order to see what the maximum movement of the nonbasic (xN)j is* such that all components of x remain feasible. This ratio test would include two cases.

1. A basic variable $(x_B)_{j\neq k}$ decreases to zero (lower bound) first.
2. The basic variable, $(x_B)_k$ increases to an integer.

Specifically, corresponding to each of these two cases above, one would compute

$$\theta_1 = \min_{i\neq k|\alpha_{j^*}>0} \left\{ \frac{\beta_i}{\alpha_{j^*}} \right\} \tag{18}$$

$$\theta_2 = \Delta_{j^*} \tag{19}$$

How far one can release the nonbasic $(x_N)_{j^*}$ from its bound of zero, such that vector x remains feasible, will depend on the ratio test θ^* given below

$$\theta^* = \min(\theta_1, \theta_2) \tag{20}$$

Obviously, if $\theta^* = \theta_1$, one of the basic variables $(x_B)_{i\neq k}$ will hit the lower bound before $(x_B)_k$ becomes integer. If $\theta^* = \theta_2$, the numerical value of the basic variable $(x_B)_k$ will be integer and feasibility is still maintained. Analogously, we would be able to reduce the numerical value of the basic variable $(x_B)_k$ to its closest integer [$_k$]. In this case the amount of movement of a particular nonbasic variable, $(x_N)_{j^*}$, corresponding to any positive element of vector $_{j'}$, is given by

$$\Delta_{j'} = \frac{f_k}{\alpha_{kj'}} \tag{21}$$

In order to maintain the feasibility, the ratio test θ^* is still needed.

Algorithm

The table 1 partition the full index set , {1, 2, ..., *n*}, ie $J_B \cup J_S \cup J_L \cup J_U$ = {1, 2, ..., *n*} and $J \ \cap J \qquad \varnothing, \quad \neq \beta$. The J_1 range of indices for integer variables is assumed to be of small, and $m + n_S + n_L + n_U = n$ *range is assumed to be small.*

The strategy implies that the ongoing issue is resolved and that it seeks an integer-feasible alternative in the near neighborhood of a continuous solution. The basic philosophy of the analysis includes the abandonment of non-basic integer variables (and hence the value of integer) and the search for limited space for basic, super basic and non-basic continuous variables, $j \notin J_I$.

The method can be generally described this way:

1. Acquire solution of the continuous relaxation as a non-linear programming problem.
2. CYCLE1: move an improper variable to an integer value by removing it from the basis at the boundary and turning it into a super-basic array, which is substituted by the previous non-basic value.

We partition, denote and define the index set as in table 1 below:

Table 1. Difene and simplex partition index sets

Name	Difene the required index sets for	Cardinality
J_B	basic variables	$\lvert J_B \rvert = m$
J_S	super basic variables	$\lvert J_S \rvert = n_S$
J_U	nonbasic variables at upper bounds	$\lvert J_L \rvert = n_L$
J_L	nonbasic variables at lower bounds	$\lvert J_U \rvert = n_U$
J_I	integer variables	$\lvert J_I \rvert = n_I$

3. CYCLE2, pass1: adapt integer-infeasible super-basic by fractional steps to meet complete integer-feasibility.
4. CYCLE2, pass2: adjust integer feasible super-basics. By conducting a high-localized neighborhood search to verify local optimality [11].

There are seven steps in cycle as follows.

Step 1. Get row i^* the smallest integer not feasible, such that $\delta_{i^*} = \min\{f_i, 1 - f_i\}$

Step 2. Perform a pricing operation

$$v_{i^*}^T = e_{i^*}^T B^{-1}$$

Step 3. Calculate $\sigma_{ij} = v_{i^*}^T a_j$

With j corresponds to

$$\min_j \left\{ \left| \frac{d_j}{\sigma_{ij}} \right| \right\}$$

Measure the maximum non-basic j movement at the upper and lower bounds. Alternatively, go to the next non-integer non-basic or super-basic j (if available). Column j^* will finally be increased by form L_B and reduced by U_B. it none If none them is heading to the next i^*.

Step 4. Solve B $_{j^*}$ = $_{j^*}$ for $_{j^*}$

Step 5. Perfrom ratio test for the basic variables in order to stay feasible due to the releasing of non-basic j^* from its upper and lower bounds.

Step 6. Exchange basis

Step 7. If row $i^* = \{\emptyset\}$ go to Stage 2, otherwise

Repeat from step 1.

CONCLUSION

Optimizing a massive water distribution system is a tricky issue. Replacement of the water pipeline plays a very crucial task in managing the inaccuracies of the water pipe, the budgeting of the infrastructure and the quality of the community service. Analyzing reliability and optimizing appropriate replacement results will improve the water pipeline replacement scheduling. This paper created a model for water pipelines to provide refined financial substitution plans to meet the needs of minimum costs and interruptions in operation.

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