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Rehabilitation Strategy for Water Networks using Risk-Performance Based Decision Making

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ABSTRACT

A proactive rehabilitation strategy is introduced based on optimizing scheduling of individual pipes for replacement, considering minimum costs and maximum performance. Performance indicators are used to identify level of a system performance that reveals severity of breakage consequences (risks). Risks are estimated by using breakage rate and consequences of the failure. In this article the performance indicators used are pressure and connectivity to identify hydraulic capacity and reliability of the system performance. A rehabilitation strategy is applied when performance indicators fall below predefined threshold, the present paper describes such an approach, which permits a judicious choice of solution along a trade-off curve between performance and cost. This strategy was applied on a real-world water distribution system, Benghazi city in Libya, the required rehabilitation is determined for the next five years (2018 to 2025) in addition, significance improvement in hydraulic performance are observed too.

Keywords:

Breakage rate. Rehabilitation. Replacement.

Risk performance Pipe. Reliability.

Water distribution system.

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1 INTRODUCTION

Reliability of water distribution systems (WDS) can be studied by considering two types of failures; mechanical failure, which refers to failures of system components, such as a pipe breakage. The second type is the hydraulic failure, which refers to failure of a system to supply users with designed demand due to pipe roughness. Both cause reduction in pressures and hence reduction in nodal demand. Knowing that performance is quantity and quality of this reliability, that a system provides under a normal and a breakage conditions as well, Kleiner Y (1997). Level of performance that reveals severity of breakage consequences (Risks) is called performance indicators. These performance indicators are used to identify level of a system reliability, which may include; hydraulic capacity and reliability to provide all regular, peak and emergency demand for water at an acceptable performance of flow rate, pressure and

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connectivity, water quality to provide safe drinking water with compliance regulatory standards and at acceptable performance level of taste, color, and odor, and socioeconomic performance to provide flow at minimum level of interruptions and service disruption of traffic and business, number and type of affected customers, S S Emil J et al. (2002). Therefore, a system is considered fail when one or more of its performance indicators fall below a predefined threshold level resulting in various risks. Predicting of failure and taking preventive action in the proper time ensures continued reliability of the system with the target performance. This "predicted" reliability is very helpful to locate, not only an effective proactive rehabilitation strategy but also in effective operation and optimal design or extension. Proactive rehabilitation strategies seek to maximize a system performance for minimizing risks resulting from failure, L Dridi, et al 2005. Quantitative risk resulting from consequences of any of pipe performance failure becomes a compulsory objective towards development of a proactive rehabilitation strategy. Risk of a failure is estimated by using frequencies and consequences of the failure. In other words, risk estimation in rehabilitation of WDS, aims to determine; how often pipe breakage might occur or probability of pipe breakage occurrence, the chance for a specified performance indicator fall below predefined threshold and the magnitude of their consequences if it happens. In order to give an expression of the total risk of a water supply system, several types of potential consequences can be considered S S Emil J et al (2002). Among several reliability measures, pressure limits, demand and discontinuity are major in evaluation network performance, particularly, hydraulic capacity or reliability and sustainability, Kleiner et al 1997. In this article, hydraulic reliability (pressure and discontinuity) and its expected consequences (risks) are identified, thus a strategy is introduced to maximizing main network performance while minimizing risk. Or in other words, optimizing scheduling of individual pipes for replacement, while considering minimum costs and maximum performance.

2 LITERATURE REVIEW

Scheduling water pipes replacement receiving wide attention from many researchers to keep reliability of systems, mitigate risks of failures and saving costs as well, e.g. and not exclusively, Kleiner 210 used fuzzy-based methods to assess post-renewal deterioration rate and subsequently make rational decisions on when to schedule a subsequent condition assessment of a pipe, when to renew a deteriorated pipe, and how to select the most economical renewal alternative, assuming it is technically feasible and appropriate. Later on, scheduling pipe replacement based on maximizing network reliability and minimizing cost is introduced by Dandy and Englehardt (2006) using a double objective trade-off. Moglia, M. at el (2006) introduced a decision support tool that performs multi-criteria analysis (criteria include hydraulics, and forecasted failures) to select pipes for renewal in short term, James Thoson 2009 developed a "Decision Support System for Distribution System Piping Renewal" to prioritize the replacement of cast iron distribution mains. Nafi, A, Kleiner, Y. 2010 proposed a method for the optimal scheduling of individual pipes for replacement, while considering practical issues such as harmonizing pipe replacement with known roadwork and economies of scale.

3 THEORIES AND METHODS

3-1 RISK EVALUATION

Risk mitigation or increasing a pipe performance can be achieved by reducing failure probability and/or its incurring consequence cost, as risk depends both on probability and cost of failure. Thus, risk can be expressed mathematically as probability of failure multiplied with cost of failure, Eq. 1.

$$Risk\ of\ failure = E\ (failureconsequence) = f\ (\text{probability of failur, costs of failure}) \quad (1)$$

Traditionally, consequences (Risks) of a pipe failure are converted to monetary values (costs). Therefore, maximizing a system performance (or minimizing risks) is not without cost. A higher level of required performance, a greater cost will be required. Providing solutions with minimum cost alternative that will facilitate maximum pipe performance, may be impossible, S. Sregrov et al, 1999. However, replacing pipes at specific time is an approach to an intelligent rehabilitation strategy that able to yield the most balanced solutions among cost and performance. Trade-off concept is an exclusive assist in selection process to find a formula that best combines such conflicting requirements, S. Park et al 2000. Measures to mitigate risk from cost side are possible but rather controversial and limited. Estimating costs of all different failure consequences (e.g., liabilities from accidents, social and business impacts) is not an easy task. Recalling direct, indirect and social costs for many failure consequences introduced by many researchers, e.g., Rajani et al. 2002, constitute a big confusion how to determine cost of human lives that are lost or exposed to injuries as a result of failure of a pipe or group of pipes in a water network, or how to evaluate cost of social or business disruption. On other hand, cost of failure is assumed implicitly time independent, which is contrary to fact. Because cost of failure is also likely to increase over time, for example, when a pipe is located in a rapidly developing area, social, business and traffic consequences cost are likely to increase too. Thus, the most commonly considered situations are where precise consequences are uncertain, but their probabilities are known. Effect of uncertainty and lack of knowledge about pricing specific failure consequences form a real barrier for risk cost based decision-making, Melita Stevens et al 2005. This uncertainty is a major reason for underestimating risk if consequences costs are underestimated, and vice versa. In fact, risk-cost based decision making which is adopted by researchers in engineering systems is influenced by what followed in economical sectors. In economical firms, evaluating failure consequences in monetary values are easily to be carried, at the same time simulation a firm performance is more hard to carry out, due to huge uncertain parameters effect the performance, some of them are not possible to model and simulate (e.g., incentives). However, in engineering systems, a simulation for different scenarios under different loads considering different uncertainties can be done in more realistic manner, thus a performance of a system can be evaluated, S. Park et al (2002). Risk in case of engineering can be accounted as a system performance deviates from its target (design). In this article, the risk is dealt with as the indicator for any deviation in operated (actual) capacity of a system from a designed capacity (function). In risk-cost based strategy, all the required is; prediction of a failure rate or probability of failure, determining the required failure consequences (Risks) to be avoided and fixing cost for each consequence, S. Park et al 2002. Risk value of each component in a system is found by multiplication of its probability of failure by cost of considered consequences, Eq.1. Risk-performance based decision making is an approach we introduce to avoid problem of converting failure consequences into monetary values, we use a fact that risk is a performance complementary (Risk = 1- Performance). In water networks, if performance of a pipe or a network decreases with time, means the risk is increased with time as well. Despite, complexity of performance determination of network pipes under number of failures scenarios, but it remains easier and more robust than determination cost of consequences under same number of failure scenarios. Therefore, the rehabilitation strategy dependent on the introduced approach is based on maximizing performance (minimizing risk) using performance indicators instead of using performance costs and minimizing pipe replacement cost. Therefore, risk, with this approach, is mitigated by minimizing failure frequency. Nevertheless, this concept may be understood as costly exorbitant. It is true unless a balance (trade-off) between risk of failure and cost to mitigate it (by pipe replacement) is found. Because failure risk of a pipe increases and its performance decreases as long as a pipe continues to age and deteriorate without renewal and thus its probability of failure (or failure frequency) increases (note that here risk = 1- performance), and at the same time, replacement cost decreases as pipe renewal is delayed, two curves similar to given in Fig.1 can be obtained. Combining both curves, an optimal time point that minimizes both costs and risks can be found. Our objective becomes; looking for optimal time of replacement a pipe with lowest cost while meeting the required performance.

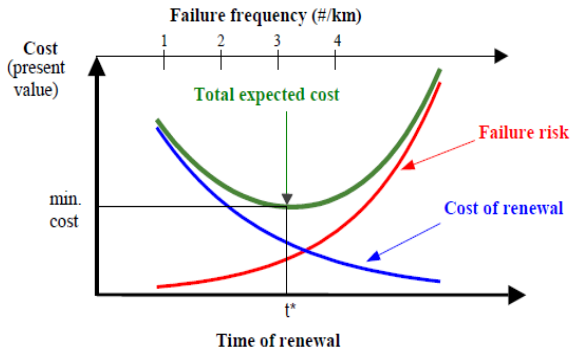

Fig 1; Failure risk vs replacement cost trade off curves, Rajani et al 2002

3-2 Pressure performance

The most frequently occurring consequences associated with pipe failure in water distribution systems is a reduction or fluctuating in pressures. Despite there are different reasons for lowering pressure, such as high elevation of area for a pressure zone serving it, or insufficient pipe or pump capacity or valve malfunctioning. However, pipe failure is considered here as a main reason for pressure reduction. Concerns of low pressures occurring in pressurized drinking water supply distribution systems are in creating an opportunity for contaminated water to enter a pipe from outside, increasing water retention time, in additional to a failure to meet demand. At the same time, high water pressure has adverse mechanical impacts on a system. Thus, it is important to maintain minimum pressure throughout a whole distribution system in order to sustain demand and microbial quality of distributed water and avoiding any rising over allowed maximum limit. For evaluation risk resulting in pressure fluctuation, becomes necessary to recognize to minimum and maximum limits of allowed pressure in a system. Determination of these limits is related to a distance over which water needs to be transported, local topographic characteristics, size of network and future extensions. Generally, minimum pressure limits depend on height of buildings in supplied area, assuming, typical buildings vary between three to five floors, this leads to a minimum pressure of 20-30 mwc (meter water column) above street level. In case of higher buildings, an internal boosting system is normally provided Nemanja 2006. Specifying a maximum allowable pressure is economically important for reducing cost of pipes resulting in additional required strength, otherwise leakages and pipe bursting are very expected. However, maximum pressure limits vary greatly (from 60 to 120 mwc) in hilly terrains. A practical method for evaluating hydraulic reliability is introduced by Cullinane (1985), he defines a nodal reliability as a percentage of time in which pressure at a node is above a defined threshold. In his eq2, he considered failure frequency and average time necessary for repair:

$$R_j = \sum_{i=1}^{k} \frac{r_{ij}\ t_i}{T} \tag{2}$$

Where R_j is hydraulic reliability of a node j, r_{ij} is hydraulic reliability of a node j during time step i, t_i is duration of time step i, k is total number of the time steps, T is length of simulation period. Factor r_{ij} takes value 1 for nodal pressure p_{ij} equal or above the threshold pressure $p_{\min}$, and $r_{ij} = 0$ in case of $p_{ij} \leq p_{\min}$. For equal time intervals, $t_i = T/k$. However, a simple modification has been conducted in terms of r_{ij} in Eq. 2 to fit our objective. Since pressure above the maximum threshold pressure could have undesirable effects as pipes burst. Therefore, Factor r_{ij} suggested taking value 0.25 for nodal pressure p_{ij} above the threshold pressure and value 1 for nodal pressure greater than or equal to minimum threshold and less than or equal to maximum threshold pressure. The reliability of the entire system consisting of n nodes can be defined as average of all nodal reliabilities:

$$R_j = \sum_{i=1}^{n} \frac{R_j}{n} \tag{3}$$

In our case study, the objective is to measure reliability or performance of a network when one specific pipe set failed or closed. Therefore, Eq. 3 is used slightly different. For our goal, within the simulation duration, we look for total time when pressure below the minimum threshold in all nodes, and the total time when the pressure above the maximum threshold and the total time when the pressure between these two limits, each phase is multiplied by its assumed performance indicator, r_{ij}, and their summation is divided by the total duration time of simulation. The result is the reliability of each pipe in the network when that pipe is closed; dividing this result by total number of pipes in the system yields the entire system reliability when one a specific pipe set closed (assumed failed).

3-3 Performance function:

In the study area, the network is designed for working pressure not less than 20 mwc and is operated for pressure not exceeding 40 mwc. This relatively small maximum pressure limit is adopted based on the deteriorated structural conditions of the pipes, in addition to the weakness of the maintenance system. Therefore, evaluation the pressure performance P is carried out in the range:

$$20\text{ m} \leq P \leq 40\text{ m}$$

3-4 Performance indicators:

Nodes having pressure values below the minimum limit 20 m yield a performance of 0%, with nodes having pressure values exceed the recommended upper limit 40 m, the performance descend to 25% and maintains in this value. Pressure greater than or equal 20 m and less than or equal 40 m yield performance of 100%. The convention adopted in this performance uses a 0 to 100% scale, with the following meanings: 100% – optimum service, 75% – adequate service, 50% – acceptable service, 25% – unacceptable service and 0% – no service.

3-5 Connectivity performance

Looped design of water networks are characterized by that water in a system flows in more than one direction and nodes can receive water from more than one side. This situation denotes by “connectivity” in which every demand node in the system is connected to any source via at least two pipes. Therefore, the connectivity is one of the reliability measures. This advantage is very useful during maintenance works, where supply to area under maintenance will not stop. However, designing large systems with high redundancy is rather difficult. Therefore, the connectivity characteristic is very important for investigating the system performance redundancy, and identifying the nodes with critical supply.

3-6 Performance function

Evaluation the connectivity performance is carried out in the range:

$$p_i = 0, \quad or \quad p_i = 1,$$

3-7 Performance indictor:

The evaluation criterion for a connectivity performance in the study network is carried out by giving 0% performance for disconnected nodes otherwise 100%. The convention adopted in this performance uses a 0 or 100% scale, with the following meanings:100% – connected service, 0% – disconnected service.

4 OPTIMAL REPLACEMENT TIME STRATEGY

Recalling the only option considered for improving a structural state and a hydraulic performance is pipe replacement. And to determine optimal time for a pipe replacement required for a proactive rehabilitation strategy, mathematically formulate a model that describes associated costs and risks is needed. In symbolic terms, the following equation for estimating levels of risk *R* of an event can be written:

$$R = 1 - P \tag{4}$$

Where; P is a system (or a component) performance. According to this definition, values of R will be in range between 0–1. Low values indicate a low level of risk, while high values, on other hand, indicate high risk. P =1 represents an ultimate case where a network is in its fully hydraulic operational functions, which means; it is functionally operated with a capacity 100% as designed capacity. However, it is elusive even for new constructed networks. Therefore, any network is eligible for a reduction in designed capacity, due to some piping failure events. The risk levels calculated by Eq.4 give an indication of consequences (Risks) of any failure event that may be caused. Specifically speaking, the major objective of every water distribution network is to maintain the operated capacity (e.g., delivery of potable water quantities) as close as possible to the designed capacity (e.g., requested demand at the minimum pressure). Like this specific objective is used in calculation of performance levels for a water network (pressure, chlorine, energy…etc). By knowing the designed capacity of a network and any reduction in that operated capacity (which can be counted as a measure of a difference between designed capacity and operated capacity as consequence of a failure), a performance of a network can be evaluated. To put it in a mathematical form, the following equation can be written:

$$P = \frac{Designed\ capacity - Operated\ capacity}{Designed\ capacity} \qquad (5)$$

Or;

$$= \frac{Operated\ capacity}{Designed\ capacity} \qquad (6)$$

As system ages, its operated capacity function decreases as a result of deterioration, thus its performance decreases as well, it can also be called actual capacity. Designed capacity, is hydraulic or quality limits, a system designed for. Performance can be calculated for each hydraulic and quality characteristics (demand, pressure, chlorine…etc.) and can be calculated for each component in a system (pipe) and for entire system as well. For example, performance of a network in respect of pressure can be measured as:

$$P_{pi} = \frac{Reduction\ in\ pressure}{minimum\ designed\ pressure} \qquad (7)$$

Where, P_{pi} is a performance of a network with respect to pressure, at a snapshot time. However, our objective is to evaluate a performance in a time extend simulation. The reduction in water pressure is actually the difference between designed and operated (or actual) pressure delivered. Reduction in a performance is, in return, rising to risk. Therefore, risk of any failure related performance would be calculated by Eq.7. Since the objective is not only to determine the optimal time of pipe replacement, but also to rank pipe replacement priority. This requires investigating all individual pipes in the network, each time one pipe set failed and the performance of the network for all assumed terms (pressure, discontinuity) is calculated;

$$\left(p_j\right)_t = \left(\frac{\sum_{i=1}^{n} p_i}{n}\right)_t \qquad (8)$$

Where; $(p_j)_t$ is performance of a pipe j due to failure of one specific pipe in a network in t year, n denotes number of investigated performances i. Pipe performance p_j is calculated for horizon plan project, in each year t, failure rate is calculated and pipe age is added 1 year. Risk caused by a pipe j R_j due to failure of a specific pipe is calculated by inference Eq. 9;

$$R_j = 1 - \left(\frac{\sum_{i=1}^{n} p_i}{n}\right)_t \tag{9}$$

5 COST OF REPLACEMENT

Term replacement cost refers to amount that a utility would have to pay to replace an asset (e.g., a pipe) at present time, according to its current worth, at its pre-loss condition. Replacement time is the time after which it is no longer economical to repair it. Therefore, replacement cost for an asset declines exponentially with time according to a predefined discount rate;

$$C_{Ri,t} = c_{it}\ e^{-rt} \tag{10}$$

Where; $C_{Ri,t}$, is replacement cost of an asset (pipe) i at time t (present value), $c_{i,t}$ is actual cost of an asset i at time of decision making (start of project), e is an exponential form of discount and r is discount rate. For simplicity, actual cost of a pipe is related to a diameter, for example, 300 mm pipe diameter costs $300 and so on, and r is assumed 3%. Because replacement cost is impacted by failure rate of an asset (as failure rate increases, replacement time decreases and replacement cost increases). Therefore, the real replacement cost of an asset becomes;

$$C_{Ri,t} = c_i\ e^{-rt} . N_b \tag{11}$$

Where; N_b is the predicted number of failures.

Using results of Eq.9 and 11, level of risk curve for each individual pipe in a network and replacement cost can be drawn as given in Fig.1. Optimal time of replacement is the minimum combine of the two curves. Since the objective is to establish the total risk values for a water network under a specific operational state (scenarios of failure), therefore, definitions should be given for total probability of collective risk-causing events and total consequences resulting from them. Those definitions were found to be convenient for this study, although variations may be made in this respect, and as appropriate.

6 METHODOLOGY

- Risk of a failure is estimated by using breakage rate (frequencies) and consequences of the failure. Therefore, how often pipe breakage might occur, or chance for a specified performance indicator fall below predefined threshold and the magnitude of their consequences, can be determined. The used breakage rate Table 1 is taken from (Bubteina et al 2011).
- A predictive model of each pipe in the system based on cleaned data that developed by (Bubteina et al 2011) was used to predict future pipe breakage for 5 years (2018 - 2023). Predicted number of breaks per year is calculated by multiplying breakage rate by length of a pipe Table 2.

- Building a hydraulic model with the help of a computer-modeling tool EPANET for an existing looped water distribution network was constructed. A model for the case study network has been prepared consisting of a group of EPANET input files describing the various steps of the analysis. In this article the problem being analyzed is the analysis of failures scenarios of the main system pipes.
- For pressure performance, a hydraulic simulation for 72 h duration (T) has been run repetitively, each time one pipe was assumed closed, and pressures in nodes are recorded. In each simulation, length of time in which pressure below 20m is determined (t_i) for each node given r_{ij} = 0, and length of time in which pressure above 40m is determined (t_i) for each node given r_{ij} = 0.25, and length time in which pressure is between 20 m and 40 m is determined for each node given r_{ij} = 1. Pressure performance of each node is calculated by using Eq. 3 and the total performance reliability is determined by using Eq.4 each time a pipe set closed. From which, contribution of each pipe in the pressure performance or effect of failure of each pipe in the system reliability can be determined

- For continuity performance, EPANET notifies a network as being disconnected in three cases; (1) if there is no way to supply water to all nodes that have demands. This can occur if there is no path of open links between a junction with demand and either a reservoir, a tank, and a junction with a negative demand. (2) When a pump is set to operate outside the range of its pump curve. This occurs if the pump is required to deliver more head than its shutoff head. (3) When the model meets negative pressures at junctions that have positive demands. This occurs when portions of the network can only receive water through links that have been closed off, indicating some problem with the way the network has been designed or operated. Therefore, the model of the case study has been run repetitively, each time one pipe set closed to investigate all pipes connectivity function.

- The case study is WDS of Benghazi city , fig 2, which consists briefly of: 419 pipes segments with total length of 373.147 km (N.B in all of following tables only sample of WDS data is shown due to its length) with different diameters (300 to 2500 mm), 36.4% of total pipes are made of 300 mm diameter and 27.4% are made of 400 mm. Uncoated steel pipes form more than half of the pipes and almost the other half made of ductile iron. About 25% of the system is more than 35 years old and about 20% is 25 years old and 30% as new as 5 years old and rest of the system is about 27 years old in average (Bubteina et al 2011), corrosion is the main problem of the pipes as well as the other components of the system. The degradation of WDS made it disable to provide safe, potable water for domestic use, adequate quantity of water at sufficient pressure for fire protection and water for industrial use as well, which created a major environment, social, economy and health problems in the city, and then their rehabilitation become a certain national devoir. The majority of numbers of breaks were found in 300-mm pipes (415 breaks) and 400-mm-diameter pipes (149 breaks) because 63% of the total network length is made of pipes of these two sizes. A total of 67% of the failures occurred for pipe ages between 32 and 40 years. However, there were breaks for all pipe ages, even for the 5-year-old pipes. With regard to the pipe materials, 58% of the failures occurred in ductile iron (average

age of 19 years) because the ductile iron pipes form about 56% of the total length of the pipes; meanwhile, 41% of the failures occurred in uncoated pipes (average age of 36 years). The breakage rate of the uncoated pipes is about 0.26 break/ km/ year, and for the ductile pipes, it is 0.20 break/ km/ year (Bubteina et al 2011).

Table 1. predicted breakage rate of each pipe for 5 years (sample of data)

	Year					
	2018	**2119**	**2020**	**2021**	**2022**	**2023**
Pipe ID	0	1	2	3	4	5
Pipe 87	0.0656	0.0693	0.0722	0.0746	0.0766	0.0784
Pipe 61	0.0503	0.0677	0.0801	0.0893	0.0964	0.102
Pipe 89	0.1557	0.1559	0.1561	0.1562	0.1564	0.1566
Pipe 80	0.0605	0.0606	0.0606	0.0607	0.0608	0.0608
Pipe 10	0.1206	0.1207	0.1209	0.121	0.1211	0.1213
Pipe 25	0.1499	0.1502	0.1504	0.1506	0.1509	0.1511
Pipe 52	0.1153	0.1155	0.1157	0.1158	0.116	0.1162
Pipe 233	0.1182	0.1184	0.1186	0.1187	0.1189	0.1191
Pipe 155	0.2312	0.2316	0.232	0.2323	0.2326	0.233
Pipe 32	0.2312	0.2316	0.232	0.2323	0.2326	0.233

Table 2. predicted number of breaks of each pipe in the system for 5 years (sample of data)

	2018	**2119**	**2020**	**2021**	**2022**	**2023**
Pipe ID	1	2	3	4	5	6
Pipe 87	0.001312	0.001386	0.001444	0.001492	0.001532	0.001568
Pipe 61	0.001107	0.001489	0.001762	0.001965	0.002121	0.002244
Pipe 89	0.006228	0.006236	0.006244	0.006248	0.006256	0.006264
Pipe 80	0.003025	0.00303	0.00303	0.003035	0.00304	0.00304
Pipe 10	0.00603	0.006035	0.006045	0.00605	0.006055	0.006065
Pipe 25	0.019487	0.019526	0.019552	0.019578	0.019617	0.019643
Pipe 52	0.017295	0.017325	0.017355	0.01737	0.0174	0.01743
Pipe 233	0.019503	0.019536	0.019569	0.019586	0.019619	0.019652
Pipe 155	0.04624	0.04632	0.04872	0.04646	0.04652	0.04893
Pipe 32	0.04624	0.04632	0.058	0.04646	0.04652	0.05825

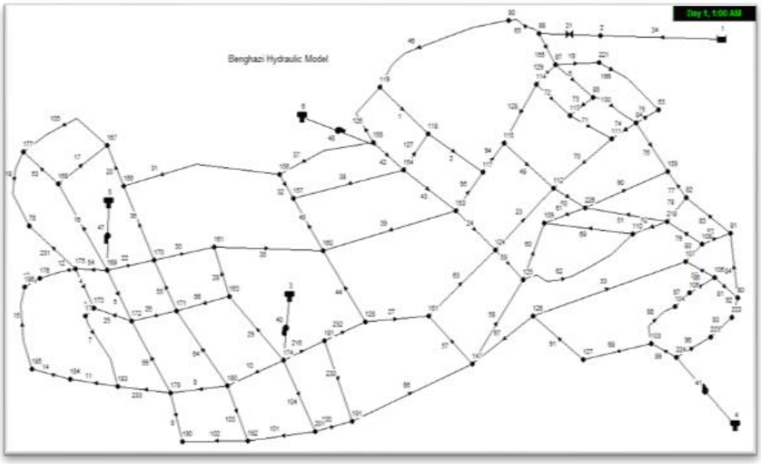

Fig. 2. Hydraulic model for Skeletonized Benghazi water distribution system (lengths are compressed so model can be seen in one screen)

7 RESULTS AND DISCUSSION

7-1 Performance index

In order to evaluate service of a system, a predefined performance evaluation function for only reliability (pressure and connectivity) is applied for each single pipe in a system. Numerical performance values for this service is determined by assuming every time a pipe closed in a system, and according to predefined performance indicators. Performance values are, thus, calculated at a pipe level and across a network. This method can show significance and influential of each pipe related to the system as a whole. To examine the service performance and how is effected by each pipe in the system, each time a pipe is set closed. Average network performance of the study year (2018) and 5 predicted years are calculated. Summary of the network performance is given in Table 3, from which, inclination of the network performance is noticeable as system ages. Generally, average system performance does not exceed 65%. On other hand, it is so clear that the system is ideal looped network, because connectivity is available for all pipes in the network, except for pipe 34 (main water carrier). As a result, water in the system flows in more than one direction and nodes can receive water from more than one side. This advantage is very useful during maintenance works, where supply to area under maintenance will not stop. In case of failure of pipe 34 (out of data sample), water will not supply to all consumers. The most sensitive pipe for pressure, beside the main carrier (pipe 34), is pipe 33 (out of data sample). This is may be due to its length (1450 m) and large diameter (400 mm). While pipe 155, its sensitivity against pressure is probably because of its location near to pipe 34. Pipe 10 and 89 are common in being located near to reservoirs and pumps and in short lengths as well, 50 m and 40 respectively. Pipe 46 and 65 (out of data sample), are common in long lengths, 1500 m and 1640 m respectively, and of large size (400 mm).

Table 3. the average system performance when a pipe is set closed (sample of data)

	Average					
Pipe ID	2018	2119	2020	2021	2022	2023
Pipe 87	0.67	0.67	0.67	0.67	0.67	0.67
Pipe 61	0.69	0.69	0.69	0.69	0.69	0.69
Pipe 89	0.66	0.66	0.66	0.66	0.66	0.66
Pipe 10	0.68	0.68	0.68	0.68	0.68	0.68
Pipe 80	0.59	0.59	0.59	0.59	0.59	0.58
Pipe 25	0.64	0.64	0.64	0.63	0.63	0.63
Pipe 52	0.64	0.64	0.64	0.63	0.63	0.62
Pipe 233	0.63	0.63	0.63	0.63	0.62	0.62
Pipe 32	0.60	0.60	0.60	0.60	0.60	0.59
…..	….	….	….	….	….	….
Average	**0.6548**	**0.6511**	**0.6483**	**0.6424**	**0.6367**	**0.6315**

7-2 Risk

Risk is calculated as a system performance deviates from its target (design). Therefore, risk is dealt with as an indicator for any deviation in operated or actual capacity of the system from the designed capacity. This risk-performance based decision making is introduced to avoid problem of converting failure consequences into monetary values. Consequently, the risk is calculated as performance complementary (Risk = 1- Performance). Each pipe in the system assumed failed at a time to evaluate the risk of the system associated with these scenarios of failures.

Table 4. Risk level of the system associated with each pipe assumed failed at a time (sample of data).

Pipe ID	2018	2119	2020	2021	2022	2023
Pipe 87	0.33	0.33	0.33	0.33	0.33	0.33
Pipe 61	0.31	0.31	0.31	0.31	0.31	0.31
Pipe 89	0.34	0.34	0.34	0.34	0.34	0.34
Pipe 10	0.32	0.32	0.32	0.32	0.32	0.32
Pipe 80	0.41	0.41	0.41	0.41	0.41	0.42
Pipe 25	0.36	0.36	0.36	0.37	0.37	0.37
Pipe 52	0.36	0.36	0.36	0.37	0.37	0.38
Pipe 233	0.37	0.37	0.37	0.37	0.38	0.38
Pipe 32	0.40	0.40	0.40	0.40	0.40	0.41

Pipes 34, 23, 80, 32, 33, 104, 103, 216 and 64 respectively are the most pipes causing risk to the network. In this sample, only 80 and 32 of data are shown.

7-3 Replacement cost

Replacement cost is amount that a utility would have to pay to replace a pipe at present time, according to its current worth, at its pre-loss condition. The replacement cost for an asset declines exponentially with time according to a predefined discount rate (3%). Therefore, the replacement cost decreases as pipe renewal is delayed. Table 5 represents the replacement cost of each pipe in the system. Because the replacement cost is a function in a pipe diameter, the highest replacement costs are for the largest pipe sizes, as shown in Table 5.

Table 5. replacement cost of each pipe in the system (sample of data)

Pipe ID	2018	2119	2020	2021	2022	2023
Pipe 87	388.1709	376.6915	365.5517	354.7413	344.2506	334.0701
Pipe 61	291.1281	282.5186	274.1638	266.056	258.1879	250.5526
Pipe 89	388.1709	376.6915	365.5517	354.7413	344.2506	334.0701
Pipe 80	388.1709	376.6915	365.5517	354.7413	344.2506	334.0701
Pipe 10	388.1709	376.6915	365.5517	354.7413	344.2506	334.0701
Pipe 25	291.1281	282.5186	274.1638	266.056	258.1879	250.5526
Pipe 52	291.1281	282.5186	274.1638	266.056	258.1879	250.5526
Pipe 233	291.1281	282.5186	274.1638	266.056	258.1879	250.5526
Pipe 155	388.1709	376.6915	365.5517	354.7413	344.2506	334.0701
Pipe 32	388.1709	376.6915	365.5517	354.7413	344.2506	334.0701

7-4 Replacement cost based number of breaks

Since replacement cost is impacted by failure rate of a pipe (as failure rate increases, replacement time decreases and replacement cost increases), replacement cost of each pipe is multiplied by pipe breaks number, and results are given in Table 6.

Table 6. multiplication of replacement cost and number of breaks for each pipe in the system (sample of data)

Pipe ID	2018	2119	2020	2021	2022	2023
Pipe 87	0.51	0.52	0.53	0.53	0.53	0.52
Pipe 61	0.32	0.42	0.48	0.52	0.55	0.56
Pipe 89	2.42	2.35	2.28	2.22	2.15	2.09
Pipe 80	1.17	1.14	1.11	1.08	1.05	1.02
Pipe 10	2.34	2.27	2.21	2.15	2.08	2.03
Pipe 25	4.66	4.52	5.36	5.21	5.06	4.92
Pipe 52	5.04	4.89	4.76	4.62	4.49	4.37
Pipe 233	4.95	4.80	5.37	5.21	5.07	4.92
Pipe 155	17.95	17.45	17.81	16.48	16.01	16.35
Pipe 32	17.95	17.45	21.20	16.48	16.01	19.46

Based on this table, optimal time of replacement of each pipe in the system can be found.

7-5 Optimal time of replacement

Recalling Trade-off curves, minimum combination of risk (Table 4) for each individual pipe in the network and the replacement cost multiplied by breaks number (Table 6) yields optimal time of replacement. Table 7 represents the combination of risk and cost multiplied by breaks number.

Table 7. optimal time of replacement (sample of data)

Pipe ID	2018	2119	2020	2021	2022	2023
Pipe 87	0.84	0.85	0.86	0.86	0.86	0.85
Pipe 61	0.63	0.73	0.79	0.83	0.86	0.87
Pipe 89	2.75	2.69	2.62	2.56	2.49	2.43
Pipe 10	1.49	1.46	1.43	1.40	1.37	1.34
Pipe 80	2.75	2.68	2.62	2.56	2.49	2.45
Pipe 25	5.02	4.88	5.72	5.58	5.43	5.29
Pipe 52	5.39	5.25	5.12	4.99	4.86	4.75
Pipe 233	5.32	5.17	5.74	5.58	5.45	5.30
Pipe 32	18.35	17.85	18.21	16.88	16.41	16.76
Pipe 155	18.29	17.79	21.54	16.82	16.36	19.83

Note: Shaded cell represents optimal year of replacement, for example pipe 87and 61, optimal time of replacement is 2018 while pipe 89, 2023 is optimal time of replacement, and so on.

It is worth notice;

1- Pipes found causing the most risks to the network (Table 4) are not necessary to be replaced in first year of the rehabilitation strategy, where, replacement cost and number of breaks are contributed to determine the optimal time of replacement.
2- A considerable amount of performance enhancement is noticeable when running hydraulic simulation with pipes assumed renewed, reaching its maximum at 2023, illustrated graphically in Fig.3.

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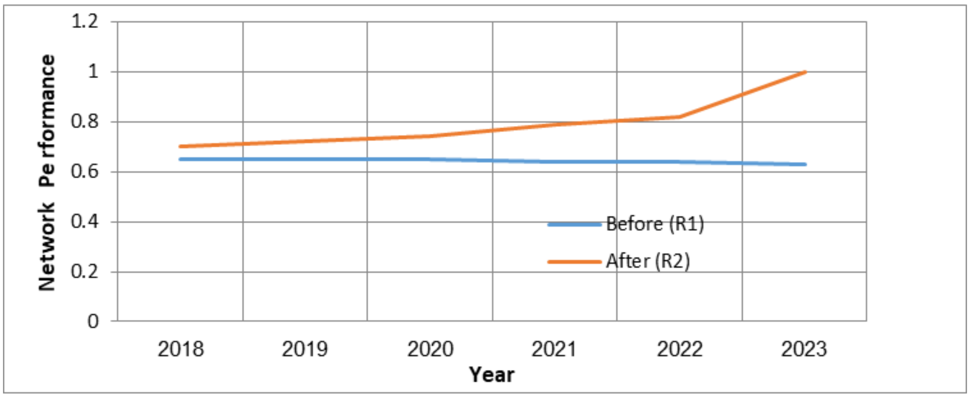

Fig. 3. performance of the network before and after the rehabilitation

8 CONCLUSION

A proactive rehabilitation decision making strategy based on maximizing main network performance (reliability) while minimizing risk was introduced. In other words, optimizing scheduling of individual pipes for replacement, while considering minimum costs and maximum performance. Performances of hydraulic reliability (pressure and connectivity) are measured by predefined threshold indicators applied on a hydraulic simulation of the network of the study area (Benghazi), and average network performance of the study year (2018) and 5 predicted years were calculated. Risk is calculated as a system performance deviates from its target (design). Therefore, risk is dealt with as an indicator for any deviation in operated or actual capacity of the system from the designed capacity. This introduced strategy is to avoid dialectical problem of converting failure consequences into monetary values. Consequently, the risk is calculated as the performance complementary. Therefore, maximizing a system performance or minimizing risks is not without cost. Where a higher level of required performance, a greater cost will be required. Providing solutions with minimum cost alternative that will facilitate maximum pipe performance reported impossible, However, replacing pipes at specific time is an approach used to the proposed rehabilitation strategy that able to yield the most balanced (trade-off) solutions among cost and performance. Trade-off concept is an exclusive assist in selection process to find a formula that best combines such conflicting requirements. Therefore, combining risk-cost curves, an optimal time point that minimizes both costs and risks were found. Optimal time of replacement each individual pipe with lowest cost while meeting the required performance were achieved.

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إستراتيجية إعادة تأهيل شبكات المياه باستخدام المخاطر - الاداء كقاعدة لاتخاذ القرار

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الملخص

تعتبر الشبكات الاصطناعية العصبية من الانظمة الديناميكية القادرة على ضبط العلاقة بين معالم مدخلات ومخرجات الانظمة شديدة التعقيد خاصة في عدم وجود الصيغة او النموذج الرياضي لهذه الانظمة.وبالتالي فهي مهمة جدا لتصميم المنظومات التي لايمكن كتابة او تحديد دوالها في صيغة رياضية. اذا ما توفرت المتغيرات الاساسية للنظام (موضوع الدراسة) حتى بدون معرفة علاقاتها ببعض فإن الشبكات العصبية الاصطناعية وباستخدامها للبيانات المتوفرة بإمكانها خلق الدالة الملائمة القادرة على التنبؤ اوتوقع العلاقات الممكنة بين تلك المتغيرات في المستقبل. في حالة تصميم او تشغيل شبكة توزيع المياه، يمكن ان نعتبر اقطار الانابيب واطوالها وعمرها ونوع التربة التي بها الانابيب ...الخ، هي مدخلات النظام ونعتبر وثوقية او اداء الشبكة هي المخرجات المرجوة من النظام. العلاقات التي تربط المدخلات مع المخرجات هي مجموعة من المعادلات المستمرة الغير خطية تمثل معادلات الفاقد في الطاقة والتصرف او التدفق مع الضغط.هذه الورقة تقدم نموذج مبني على الشبكات العصبية الاصطناعية يوفر طريقة للتنبؤ باحتمالية اى الانابيب الاكثر عرضة للكسر في الشبكة ، وبالتالي يمكننا من وضع استراتيجية استباقية للصيانة (قبل وقوع الكسر المحتمل)، ايضا من خلال هذا النموذج يمكن تحديد اولويات الصيانة والوقت المناسب لهذه الصيانة والعوامل الاكثر تأثيرا في حدوث الكسر.تم تطبيق هذا النموذج على شبكة توزيع مياه بنغازي.

الكلمات الدالة:
معدل الكسر.
اعادة التأهيل.
المخاطر.
الأنابيب الوثوقية.
نظام شبكة توزيع المياه.

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