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MAXIMIZING THE BENEFITS FROM WATER AND ENVIRONMENTAL SANITATION

Reliability assessment of the Nonsan distribution network by the method of Ozger

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The importance of water distribution network reliability is continuously growing nowadays in South Korea in order to improve the level of service to the consumers. The distribution network in the city of Nonsan occasionally experiences insufficient pressures and water quality problems. The reliability assessment of this network was performed according to the method of Ozger. The computer programme developed with EPANET Toolkit functions in Visual C++ language based on this method, conducts the Pressure-Dependent Network Analysis (PDNA). For more realistic assessment, the model network was analysed under the assumption of single pipe failure and with extended-period simulations for various 'what-if' scenarios. Prognosis of the reliability was established by using the life distribution models, which can describe an increase of pipe failure rates. According to the results, the reliability of the system is presently considered as satisfactory but the deterioration of the situation can be expected within a period of 10-15 years.

Background

Water distribution network performance has been a high concern in the present South Korea. Most of the water companies do not recognize the importance of a reliable water distribution system due to both lack of funds and unskilled engineers. Such a result has caused occasionally low level of service to the consumers, which gives birth to various complaints such as low pressures and water quality problems. In addition, the reactive maintenance demands too much time and money. That is because the water supply sector in South Korea lacks a proper water management program and is still failing to realize the importance of reliable water distribution. The problem is amplified by the fact that a generally acceptable method and procedure for reliability assessment in water distribution network practically does not exist, and could therefore not be adopted country-wide.

This study has been initiated in attempt to investigate the present state of the art regarding the reliability analysis and propose a workable approach based on available operational data. It focuses on reliability assessment by using the algorithm proposed by Ozger and Mays (2003), which calculates the reliability from the available demand fraction (ADF) under the minimum required service pressure. This reliability performance index expresses a ratio of the available- and design (i.e. originally specified) demand and is easy to understand as the basic principle. Based on this method, a computer programme was developed which works in combination with widely used EPANET2 software developed by US Environmental Protection Agency (Rossman, 2000). The model of the distribution network of the city of Nonsan was taken as a pilot, given reasonably good information about the system layout and demand.

Network reliability by the method of Ozger

Basic theory and principle

In principle, the method of Ozger can be applied with any conventional network modelling software while doing manual modification of input data. Nevertheless, even for small networks of a few dozen pipes, this process becomes extremely lengthy. The need to automate it is therefore a 'must', which has been done in this study by developing so called 'Pressure-Dependent Network Analysis' (PDNA) programme written in Visual C++ language by applying the EPANET Toolkit functions available apart form the main software. The PDNA programme is based on the Demand-Driven Available-Demand-Fraction (DD-ADF) method of Ozger and has the flowchart as shown in Figure 1. Both the nodal- and the system reliability are judged by the demand that is available during the period when the nodal pressure is below the threshold value for a variety of demand scenarios and failure of different system components. This demand is always reduced compared to the demand originally specified in the network nodes.

As noted by many researchers (Cullinane er al., 1992, Gargano and Pianese, 2000; Tanyimboh et al., 2001; Shinstine et al., 2002), the reliability concept is normally meant for non-repairable components, in which the component has to be replaced after it fails. Nevertheless, many of the components in water distribution systems, such as pumps and pipes, are generally repairable and can be put back into operation. It is therefore more appropriate to use the concept of 'availability'. Ozger considers two types of availability: mechanical availability and hydraulic availability.

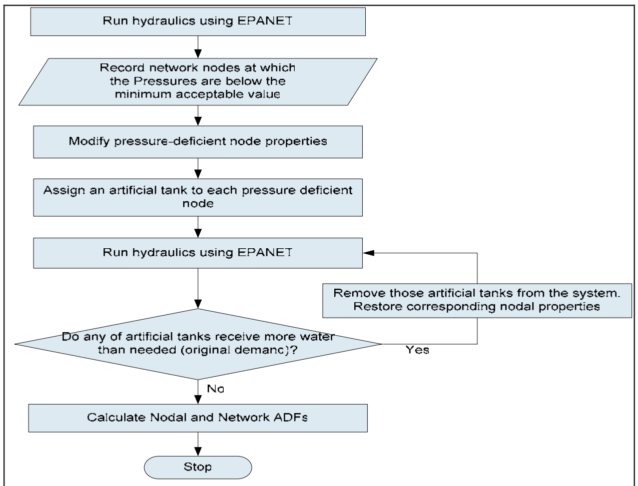

Figure 1: Flowchart of DD-ADF Algorithm

Source: Ozger

Mechanical availability

In repairable systems, two types of distribution are considered: failure distribution and repair distribution. The failure distribution describes a time it takes for a component to fail, while the repair distribution describes a time it takes to repair the component. Mathematically, the mechanical availability (MA) of a component is expressed as follows:

$$MA_i = \frac{MTTF_i}{MTTF_i + MTTR_i} \tag{1}$$

where $MTTR_i$ is the mean time to repair the i^{th} component and $MTTF_i$ is the mean time to failure, which is given by

$$MTTF_i = \frac{1}{\lambda_i \cdot L_i} \tag{2}$$

where L_i is the i^{th} pipe length and λ_i is the expected number of failures per year per unit length of the pipe. Consequently, the mechanical unavailability (MU) is the probability that a component is not operational; it is expressed as:

$$MU_i = 1 - MA_i = 1 - \frac{MTTF_i}{MTTF_i + MTTR_i} \tag{3}$$

For the entire water distribution system, the mechanical availability is described as the probability that all the components are in operation:

$$MA_s = \prod_{i=1}^{n} MA_i \tag{4}$$

where n is total number of components. At the same time, the probability of a failure of the i^{th} component and all other components remaining fully functional is given as:

$$u_i = MA_s \cdot \frac{MU_i}{MA_i} \tag{5}$$

Similarly, the event of a simultaneous failure of two components (e.g., the i^{th} and the k^{th}) is given by:

$$u_{ik} = MA_s \cdot \frac{MU_i}{MA_i} \cdot \frac{MU_k}{MA_k} \tag{6}$$

Hydraulic availability

The hydraulic availability (HA) deals with the key objective of water distribution system to deliver a specified quantity of water to a specific location at required time and under desired pressure. Ozger uses the available demand fraction (ADF) to describe the hydraulic reliability. The ADF index can be expressed as follows:

$$ADF_i = \frac{Q_i^{avl}}{D_i} \tag{7}$$

where ADF_i is available demand fraction at node i, Q^{avl} is available demand at node i and D_i is the total demand allocated to node i. The ADF of the entire network at particular moment is then given as:

$$ADF_{net} = \frac{\sum_{all\,nodes} Q_i^{avl}}{\sum_{all\,nodes} D_i} \tag{8}$$

Similarly, the ADF for a certain time interval is:

$$ADF_{net} = \frac{\sum_{t=1}^{nt} \sum_{all\,nodes} Q_i^{avl}}{\sum_{t=1}^{nt} \sum_{all\,nodes} D_i} \tag{9}$$

where nt is the number of time steps that describe the demand pattern.

c) Nodal and system reliability/availability

The hydraulic availability depends greatly on the availability of the mechanical components. Therefore, the mechanical availability must be explicitly considered in it. For assessing the nodal and system reliability, Ozger follows the work of Shinstine et al. (2002) with the available demand fraction in place of the hydraulic availability. The system reliability can be expressed as follows:

$$R_s = 1 - \sum_{i=1}^{np} (1 - ADF_{net}^{i}) \cdot P_i \tag{10}$$

where ADF_{net} is the network available demand fraction resulting from the failure of pipe i, P_i is the probability of the failure of that pipe, and np is the total number of pipes in the network. In the above equation, P_i is determined by using the Poission probability distribution as shown below:

$$P_i = 1 - e^{-\beta_i} \text{ and } \beta_i = \lambda_i \cdot L_i \tag{11}$$

where β_i is expected number of failures per year for pipe i, λ_i is expected number of failures per year per unit length of pipe i, and L_i is the length of the pipe. Considering the first and second order failures, the system availability will be as follows:

$$A_s = ADF_{net}^{0} \cdot MA_s + \sum_{i=1}^{np} ADF_{net}^{i} \cdot u_i + \sum_{i=1}^{np-1} \sum_{k=i+1}^{np} ADF_{net}^{ik} \cdot u_{ik} \tag{12}$$

where ADF^0 is available demand fraction with fully functional network, ADF^i is available demand fraction after the failure of pipe i, and ADF^{ik} is available demand fraction after the simultaneous failure of pipes i and k.

Failure rate function

The failure rate function λ is the most important measure of component reliability. Many repairable systems, including pipes, typically have a 'bathtub'-shaped failure rate function, as shown in Figure 2 (Neubeck, 2004). The total lifetime of a component can be split in three intervals. Early life failure (λ is decreasing) is normally caused by substandard weak specimens, poor construction practices or poor quality control. Useful life failure (λ is constant) is caused by unpredictable sudden stress of the components. The frequency of failure is almost constant in this period. Finally, the wear-out failure (λ is increasing) is caused by the component ageing. During this period, the failure rate will exceed a certain maximum, and it will become cost efficient to replace the component than to repair it.

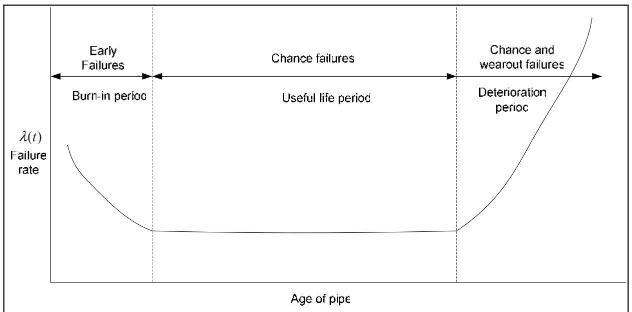

Figure 2. Pipe failure rates as a function of age

Source Neubeck

If the repair rates are fairly constant, the successive repair can be modelled as a renewal process, which characterizes repairable systems. The Homogeneous Poisson Process (HPP) model is appropriate for most of renewable systems. Here, the failure rate is assumed to be constant with time, as illustrated in Equation 11. Hence:

$$\lambda(t) = \lambda_o \tag{13}$$

where λ_0 is the assumed number of failures per year per unit length of a pipe. The early failures and wear-out failures can be modelled using the Non-Homogeneous Poisson Process (NHPP). The NHPP allows the failure rate to vary with time. This implies that the intervals between the failures are neither independent nor identically distributed (Tobias and Trindade, 1995 and Watson at al, 2001). Two commonly applied time-dependent models for the mean cumulative repair function of an NHPP are the Power Relation model and the Exponential model (Tobias and Trindade, 1995). The cumulative repair function for the Power Relation model is given by:

$$M(t) = at^b \tag{14}$$

where M(t) is the expected number of failures between time zero and t, a and b are empirically determined parameters from the pipe break records. The corresponding failure rate function is:

$$\lambda(t) = \frac{dM(t)}{dt} = abt^{b-1} \tag{15}$$

A deteriorating process is the one with 0<b<1 while the process is improving for b>1. According to the Exponential model, the failure function becomes:

$$\lambda(t) = e^{c+bt} \tag{16}$$

where c and b are empirically determined parameters from the pipe break records, and t is the time counted from the base year.

Sample network simulation

To test the method, the sample network on Figure 3 taken from Khomsi et al. (1996), was chosen. The Hazen-Williams coefficient and length of all pipes were set at 130 and 1000 m, respectively. The nodal elevations and required heads are shown in Table 1. The diameter of pipe 3 is 100 mm, pipes 7 & 8 = 150 mm, pipes 4 & 5 = 200 mm, pipes 1 & 2 = 250 mm and pipe 6 = 300 mm. The mean pipe-failure probability is shown in Table 2. A synthetic demand pattern as in Figure 4 was used for the calculation. The threshold pressure and mean-time-to-repair were 20 mwc and one day, respectively.

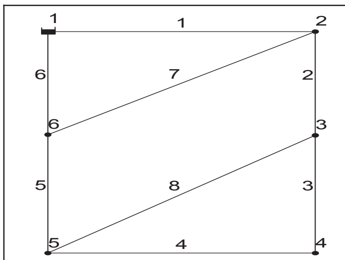

Figure 3: Layout of sample network

Source: Khomsi et al

Table 1: Nodal demands, elevations and heads

Node	Mean demand (l/s)	Ground Level (m)	Threshold Pressure (mwc)	Required Head (m)
1	-150	200(Source)		
2	20	158	20	178
3	30	158	20	178
4	40	148	20	168
5	30	155	20	175
6	30	174	20	194

Table 2. Mean pipe failure probabilities by diameter

Dia. (mm)	Failure rate (breaks/km/year)	Dia. (mm)	Failure rate (breaks/km/year)
100	0.3288	300	0.0390
150	0.1708	350	0.0390
200	0.0701	400	0.0259
250	0.1350	450	0.0259

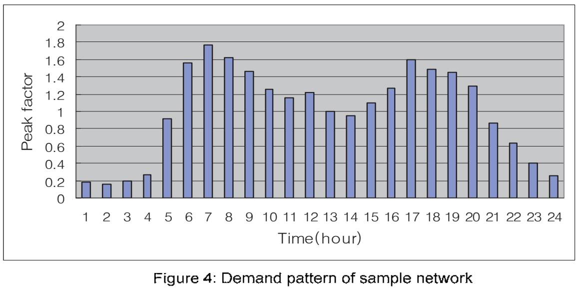

Figure 4: Demand pattern of sample network

Source: Author

Illustration of the program simulation process

To calculate the ADF index, the simulation starts with the network model without pipe breaks and is then repeated under assumption of a single pipe break at a time; the number of these simulation runs equals the total number of pipes in the network. The example in Figure 5 illustrates the procedure in case the burst of pipe 6 occurs at 13.00 hours (the hourly peak factor at that moment is 1.005).

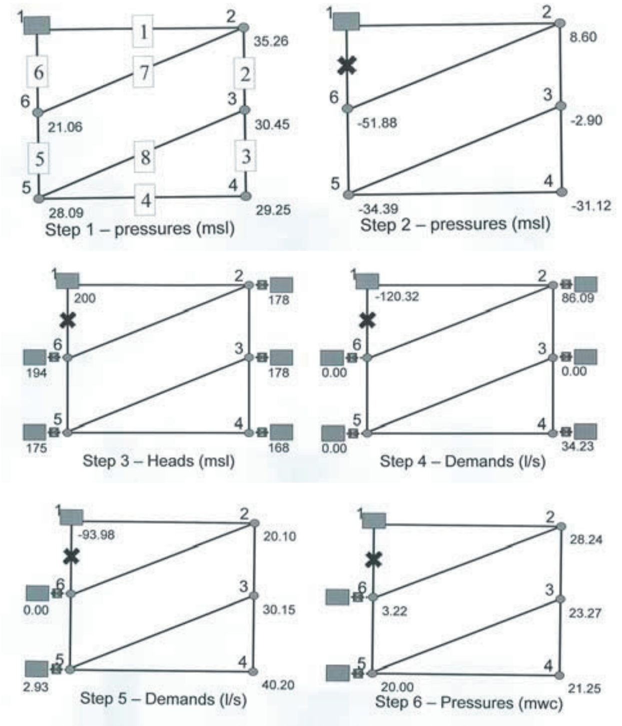

Figure 5 – PDNA programme simulation principles

Step 1: A fully functional network with no pipe failures is analysed. All nodes satisfy the threshold (i.e. the minimum acceptable pressure) of 20 mwc (MAP).
Step 2: Simulation of the system without pipe 6 in operation shows all nodes to have the pressure below MAP.
Step 3: All nodes with the pressure below MAP (in this case all in the network) are transformed into a system consisting of an artificial reservoir with the head equal to the required head from Table 1. This reservoir is connected with a short pipe containing a non-return valve to prevent the backflow from the reservoir. To assess the actual demand, the original nodal demand has been set to zero.
Step 4: After the modification of the network, the model simulation runs again. If one or more artificial reservoirs have received more water than the original demand of the corresponding node, those artificial reservoirs are to be removed from the network and original demands at the corresponding nodes are restored. For example, the demand of artificial reservoir 7 (86.09 l/s) has exceed the original demand of node 2 (20.10 l/s) = 20 l/s * 1.005) and this reservoir can be removed from the network.
Step 5: With the original demand of node 2 restored the simulation is run again. In nodes where the reservoirs receive less water than the original nodal demand, this figure shows the degree of the demand reduction. In this case, the total available demand is 93.38 l/s, whilst the original actual demand was 150.75 l/s. Therefore, the overall available demand fraction (ADF) is 61.94% (93.38/150.75). The original demand of nodes 2,3, and 4 is fully satisfied, whilst the demand of node 5 will be partially satisfied (ADF = 9.72 %). Node 6 will receive no water as a result of the pipe burst (ADF = 0 %)
Step 6: In all nodes that are not modeled as artificial reservoirs, the actual demand will per definition match the original demand, which is confirmed by the pressures that are above the MAP.

The above illustration of the process gives immediate impression about its complexity if it is to be repeated for the entire simulation period and the entire network. Applying it on any larger network by modifying the model input data manually makes it so time-consuming that the whole process becomes extremely cumbersome.

Reliability/Availability of sample network

The PDNA programme uses the EPANET2 software for network hydraulic calculations. Next to the EPANET input file, the PDNA manipulates three other input files: the file with the pipe failure rates, minimum nodal pressures and minimum repair times. Various output files are produced including the files with intermediate calculations. The generated results can further be processed by using the spreadsheet such as in Table 3.

The results of the test network showed that the calculated availability factor (0.8789), discussed in Equation 12, is higher than the reliability factor (0.7742), discussed in Equation 10, which is normal knowing that the availability assumes possibility of repair of the failed system component. Also logical is that the higher demand in the system inflicts the lower values of the availability and reliability factors, as Figure 6 shows during particular hours of the day. Figure 7 illustrates the diurnal profile for the nodal reliability. The nodal reliability values are generally higher at low demand times and are lower at peak times. For example, at 7 a.m. and 5 p.m. at the highest demands, all the nodal reliabilities experience their lowest values. Especially, nodes 4, 5 and 6 are sensitive in this respect.

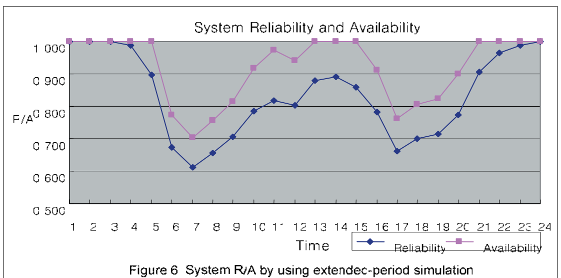

Figure 6 System R/A by using extended-period simulation
Source Author

Table 3: Reliability and availability calcualtion at 13 hour

System R/A

Pipe ID	D(m)	Length(m)	λ [1] (breaks/km/year)	$MTTF_i$ [2] (year)	β_i [3] (breaks/year)	p_i [4]	MA_i [5]	MU_i [6]	u_i [7]	ADF_{net} [8]	$(1-ADF_{net})*p_i$ [9]	$ADF_{net}*u_i$ [10]	R [11]	A [12]
No breaks						0.06979	0.99693			1.0000	0.00000		0.0000	
1	0.250	1000	0.13505	7.405	0.13505	0.12633	0.99963	0.00037	3.689E-04	0.642	0.04523	2.368E-04	0.0452	0.0002
2	0.250	1000	0.13505	7.405	0.13505	0.12633	0.99963	0.00037	3.689E-04	0.760	0.03032	2.803E-04	0.0755	0.0005
3	0.100	1000	0.328865	3.041	0.32887	0.28026	0.99910	0.00090	8.982E-04	1.000	0.00000	8.982E-04	0.0755	0.0014
4	0.200	1000	0.07008	14.269	0.07008	0.06768	0.99981	0.00019	1.914E-04	0.808	0.01299	1.547E-04	0.0885	0.0016
5	0.200	1000	0.07008	14.269	0.07008	0.06768	0.99981	0.00019	1.914E-04	0.760	0.01624	1.455E-04	0.1048	0.0017
6	0.300	1000	0.039055	25.605	0.03906	0.03830	0.99989	0.00011	1.067E-04	0.619	0.01459	6.603E-05	0.1194	0.0018
7	0.150	1000	0.17082	5.854	0.17082	0.15703	0.99953	0.00047	4.666E-04	1.000	0.00000	4.666E-04	0.1194	0.0022
8	0.150	1000	0.17082	5.854	0.17082	0.15703	0.99953	0.00047	4.666E-04	1.000	0.00000	4.666E-04	0.1194	0.0027
								0.00307			0.11938	0.00271		
													0.8806	0.9996

Node (6) R/A

Pipe ID	D(m)	Length(m)	λ [1] (breaks/km/year)	$MTTF_i$ [2] (year)	β_i [3] (breaks/year)	p_i [4]	MA_i [5]	MU_i [6]	u_i [7]	ADF_{net} [8]	$(1-ADF_{net})*p_i$ [9]	$ADF_{net}*u_i$ [10]	R [11]	A [12]
No breaks						0.06979	0.99693			1.0000	0.00000		0.0000	
1	0.250	1000	0.13505	7.405	0.13505	0.12633	0.99963	0.00037	3.689E-04	0.378	0.07858	1.394E-04	0.0786	0.0001
2	0.250	1000	0.13505	7.405	0.13505	0.12633	0.99963	0.00037	3.689E-04	1.000	0.00000	3.689E-04	0.0786	0.0005
3	0.100	1000	0.328865	3.041	0.32887	0.28026	0.99910	0.00090	8.982E-04	1.000	0.00000	8.982E-04	0.0786	0.0014
4	0.200	1000	0.07008	14.269	0.07008	0.06768	0.99981	0.00019	1.914E-04	1.000	0.00000	1.914E-04	0.0786	0.0016
5	0.200	1000	0.07008	14.269	0.07008	0.06768	0.99981	0.00019	1.914E-04	1.000	0.00000	1.914E-04	0.0786	0.0018
6	0.300	1000	0.039055	25.605	0.03906	0.03830	0.99989	0.00011	1.067E-04	0.000	0.03830	0.000E+00	0.1169	0.0018
7	0.150	1000	0.17082	5.854	0.17082	0.15703	0.99953	0.00047	4.666E-04	1.000	0.00000	4.666E-04	0.1169	0.0023
8	0.150	1000	0.17082	5.854	0.17082	0.15703	0.99953	0.00047	4.666E-04	1.000	0.00000	4.666E-04	0.1169	0.0027
								0.00307			0.11688	0.00272		
													0.8831	0.9997

Note: 1): pipe failure rate, 2): caluated by equation(2.14), 3): calculated by equation (2.3), 4): calculated by equation (2.3)
5): calculated by equation(2.13), 6): calculated by equation(2.16), 7): calculated by equation(2.18)
8): calclated by the program(equation: 2.40 and 2.41) , 9): use for reliability calculation, 10): use for availability calculation
11): reliability calculation by using the equation (2.38), 12): availability calculation by using the equation (2.39)
: the results of the above-mentioned simulation process

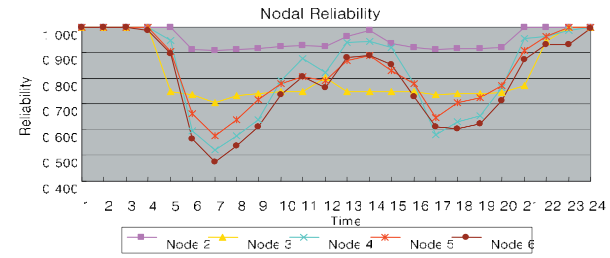

Figure 7: Nodal reliability by using extended-period simulation

Source: Author

Modelling of the Nonsan Network

General information

Water supply system in Nonsan municipality is composed of two water treatment plants and intake facilities, eight reservoirs (tanks) and nine pumping stations. By the end of 2003, the water supply system of Nonsan supplied approximately 78 thousand people which account for 56.2% of the total population in the municipality. Since the policy for water distribution has been promoted focusing on the town areas, the rate of water supply in those areas increased more than 90%, whilst the suburbs (called myeons) are covered with small-scale supply. Table 4 shows the general information in each of the areas (KOWACO, 2003).

Table 4: General information of the Nonsan water supply system

District	Total Population (person)	Served population (person)	Supply rate (%)	Facilities capacity (m3/d)	Com-sumption (m3/d)	Unit consumption (lpcpd)
Kangkyungeup	14,143	13,973	98.8	-	6,679	478
Yeonmueup	20,831	15,282	73.4	-	6,936	454
Yeonsanmyeon	8,288	1,453	17.5	2,200	517	356
Ugnjinmyeon	5,391	1,368	25.4	-	480	351
Choiunmyeon	3,768	1,662	44.1	-	587	353
Town area	44,595	44,262	99.3	23,200 (21,600)	21,858	394
No supply(8)	40,984	-				
Total	138,000	78,000	56.2	47,000	37,057	482

Study area

Amongst several areas in Nonsan, the town area was selected for the reliability analysis in this study. In the town area, 44,262 persons are supplied which is 56.7% of the overall supplied population in the municipality, registered in 2003. There are two sources: the Nonsan riverbed infiltration water and the Geum river (surface water) operated by the Korea Water Resource Cooperation (KOWACO). The raw water from the Geum river is treated in the Seoksung treatment plan for the consumers of several other areas next to the Nonsan municipality. The Nonsan riverbed infiltration water is transported to the Nonsan plant and treated to produce portable water. The reservoirs in the network comprise the Bonghwa and Nedong next to the reservoir in the Nonsan treatment plant. The Bonghwa reservoir has been closed because of the lack of pressure, whilst Nedong reservoir receives the treated water that is supplied from the Nonsan plant. This reservoir is located at the highest area and its high water level (HWL) is 66.6 msl and its capacity is 1,500 m^3. The water depth to be maintained in the Nedong reservoir ranges from 3.0 m to 4.5 m and the Nedong pumping station is operated in relation to this reservoir. Nonsan treatment plant has a clear water reservoir of the capacity of 6,425m^3.

Water consumption determined from the billing records indicates high levels of unaccounted for water (50.41%), which is caused by high leakage levels, inaccurate water meters and illegal connections. The overall length of the distribution network is 202.8km, wherefrom the town area takes about 40%, or 93.4km. The diameters of the main distribution pipes range between 40 mm and 600 mm. These are made of several materials, such as cast iron (CIP), ductile iron (DCIP), steel (SP), PE and PVC. The oldest pipes in the system are 34 years of age. The total length of pipe diameters and materials present in the system is given in Table 5.

Table 5: Pipe length and materials in Nonsan town area

Diameter	CIP	DCIP	HI-3P	PE	PVC	SP	Total
40	190						190
50				196	129		325
75	439	8			226		673
80	8,520	247	56	1,967	3,427		14,217
100	3,583	1,234	584	6,738	4,024		16,163
150	6,769	3,591	1,088	7,239	6,289		24,976
200	6,680	1,404		8,290	1,012		17,386
210				368			368
250	4,616	698		2,161			7,475
300	3,680	210			176		4,066
350	280	2,229					2,509
400	2,822						2,822
500						1,237	1,237
600	483					508	991
Total	38,062	9,621	1,728	26,959	15,283	1,745	93,398

The pipe burst records were not available in the Nonsan municipality that runs the local water distribution company. The pipe failure rates for the model application were acquired by statistical analysis throughout the comparison between the several pipe break data from the literature review and the general pipe break data of the KOWACO water mains. The results shown in Table 6 are therefore an approximation used for the purpose of this study and are to be verified in practice.

Table 6. Pipe failure rate for model application

Dia. (mm)	Failure rate (breaks/km/year)	Dia. (mm)	Failure rate (breaks/km/year)
75	0.3430	300	0.1018
80	0.3339	350	0.0777
100	0.2997	400	0.0593
150	0.2288	450	0.0453
200	0.1747	500	0.0346
250	0.1333	600	0.0201

Model building

The fieldwork data collection in the Nonsan network took place in period November - December 2004. After the model building procedure was completed, the model calibration was conducted by comparing the calculated pressures with the field measurements. The final model used in the reliability analysis consisted of two sources, three reservoirs, 521 pipe, 407 demand nodes, and two pumps; the layout is shown in Figure 8.

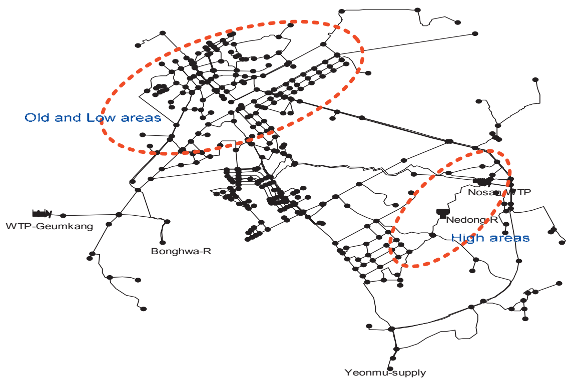

Figure 8: EPANET layout of the Nonsan network

Source: Author

The head loss formula of Hazen-Williams was used in the calculations, with the roughness coefficients categorized by the pipe age. The pumping stations in the system consist of units of uniform size with duty heads of 45 mwc and duty flows of 246 m^3/h.The diurnal demand pattern was developed by the statistical analysis method (moving average) using the trend lines from the flow measurements in the water distribution network proceeded by the Nonsan municipality in 1998. The result is shown in Figure 9; the peak factors are in the range of 0.735 to 1.312.

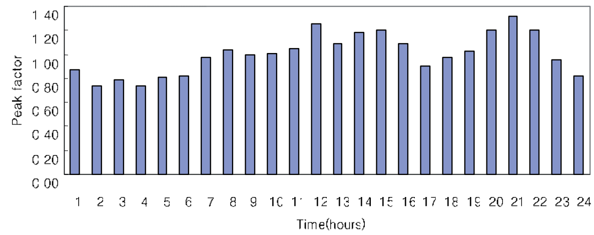

Figure 9: Diurnal Demand pattern of the Nonsan network

Source: Author

Simulation running conditions

The PDNA computer simulations were run for a period of 24 hours. The first run was done for the entire system with all components available and then the pipes were eliminated, one at a time, for each new simulation. The results obtained from the PDNA programme were evaluated and the nodal and system reliability/availability were analyzed from the spreadsheet tables such as the one shown in Table 3. The simulation conditions are shown in Box 1.

Box 1: Simulation conditions

- The pipe failure is considered a single pipe break
- The minimum required pressure is 15 mwc (Korea standard: 15~20 mwc)
- The water consumption is assumed to be the same with original water consumption when pipes fail.
- The mean-time-to-repair period is assumed at 1 day
- The pipe break rate follows the results in Table 7

The reliability assessment was conducted for both steady-state simulation and extended-period simulation. The former was based on the average demand during 24 hours. On the other hand, the latter considered the demand of each node considering the peak factor as introduced in Figure 9. Moreover, a number of 'what-if' scenarios were simulated to check the sensitivity of the reliability and availability factors. Finally, the future reliability was simulated by using the life distribution models.

Discussion of the results

The results of the model simulation are shown in Table 7 and Figure 10. From these results, the reliability in case of the extended-period simulation is lower than this of the steady-state simulation. As expected, the reliability of a high demand hour (21 hour) is lower than the one of other hours. At hour 21, the reliability shows a sudden drop and relatively low value (0.3141) due to the high demand at that moment. This means that the Nonsan network becomes increasingly sensitive to the water quantity to be supplied above a certain specific demand.

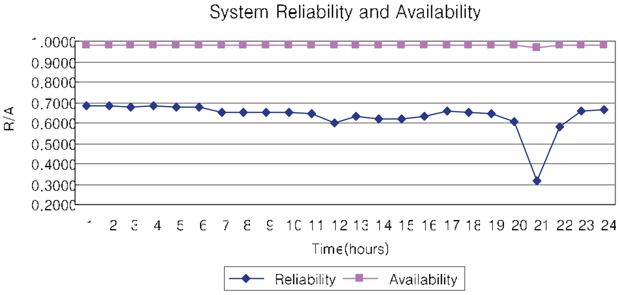

Figure 10: System reliability and availability during 24 hours

Source: Author

Table 7. System reliability and availability

Steady-state simulation		Extended-period simulation	
Reliability	Availability	Reliability	Availability
0.65257	0.98219	0.62014	0.98116

Four different 'what-if' scenarios were further simulated: threshold pressure variation, repair time variation, demand variation and roughness coefficient variation. These results are summarised in Figure 11

1) Threshold pressure variation

The increase of the minimum required pressure means that the reliability and availability of the network will decrease. For example, for the threshold pressure of 23 mwc, the reliability and availability factors were calculated at 0.125 and 0.947, respectively.

2) Mean-Time-To-Repair variation

Because the mean-time-to-repair factor (MTTR) relates to a repairable system, the reliability of the model network remains constant, but the availability of the network will decrease with the increase of MTTR.

3) Demand variation

The demand variation analysis was performed for the Nonsan network assuming that the demand increased throughout the system and each node was assigned an even portion of that increase. For example, when the demand increases for 20% (multiplier: 1.2), the reliability and availability of the model network will decrease to 0.495 from the initial 0.62 and to 0.935 from the initial 0.981, respectively

4) Roughness coefficient variation

To simulate the effects of pipe ageing, the Hazen-Williams friction factor was altered globally. The roughness coefficient was decreased by 5 to 20 %. As a result, the reliability and availability were decreased compared to the original condition. Especially, if the roughness coefficient is reduced by 20% due to the pipe ageing, the reliability of the model network would be seriously reduced.

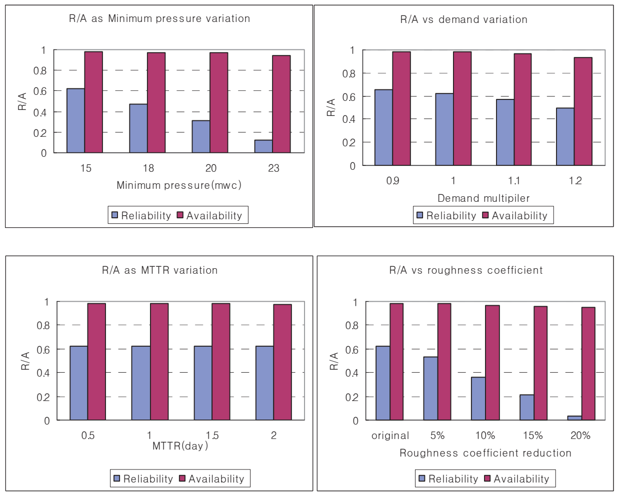

Figure 11: Several simulation results (what-if scenarios)

Source: Author

For the reliability analysis in future, the two pipe groups were defined. Based on the generated burst records, the first group that was modelled using the homogenous Poisson process (HPP) included diameters of 500mm and higher. Pipes with diameters of 450 mm or lower were modelled by applying the exponential model of the non-homogenous Poisson process (NHPP). The model parameters c and b for the exponential model were assumed at -4.83 and 0.24. Such a choice is somewhat arbitrary and was selected to fit the general tendency of smaller pipes having higher break rates. Figure 12 shows the results of the reliability/availability after 10/15 years. These results can help to determine the rehabilitation strategy in the network that can improve the reliability. E.g. assuming two different target levels of the network availiability at 0.97 and 0.94, suggests that a rehabilitation work would be needed in about 11 and 14 years from now.

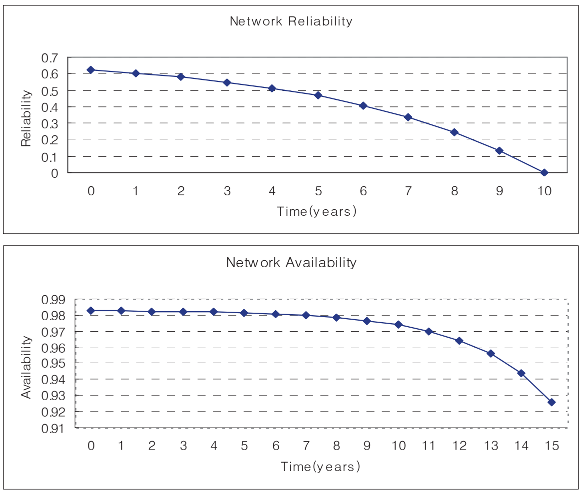

Figure 12: R/A simulation in future by using the deteriorating pipe groups

Source: Author

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