Proceedings of the 9th U.S. National and 10th Canadian Conference on Earthquake Engineering
Compte Rendu de la 9ième Conférence Nationale Américaine et
10ième Conférence Canadienne de Génie Parasismique
July 25-29, 2010, Toronto, Ontario, Canada • Paper No 16

SEISMIC RISK ASSESSMENT AND MITIGATION DESIGN FOR AN EXISTING LIFELINE

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ABSTRACT

In order to keep the existing lifeline network system at a favorable seismic performance level, it is necessary to carry out retrofitting activities. This study proposes a seismic risk assessment method for the existing deteriorated lifeline network system based on the probability of system performance failure. Numerical simulations are carried out for the existing water distribution network system for several seismic investment strategies to support the decision-making of seismic disaster mitigation planning. Effective planning of seismic retrofitting activities and disaster mitigation for the existing lifeline system can be realized using the newly developed assessment method.

1. Introduction

Since the existing lifeline network system is responsible for the lives and health of its users as well as social benefit, it is required to keep the supply service functioning as much as possible even when a severe seismic event occurs. It is therefore necessary to carry out retrofitting activities to keep the deteriorated existing system at a favorable seismic performance level. In order to plan effective retrofitting of the system for seismic disaster mitigation, it is necessary to develop a procedure to quantitatively evaluate the effect of partial retrofitting on the fragility of the existing lifeline network system. There are some indexes for evaluating the seismic performance of the lifeline system, such as structural failure, performance failure and economic loss, and they are closely interrelated. Especially, the supply damage rate after an earthquake is the most suitable index for evaluating the seismic performance of the existing lifeline network system from the viewpoint of the system users. However, since the network system normally has a hierarchical structure with a huge number of links and nodes, it is not simple to estimate the effect of partial retrofitting on the seismic performance such as loss of service.

In this study, a simple model is used to estimate the relation between the probability of structural failure and loss of service without using the topological characteristics of the network system. A seismic assessment procedure for the existing lifeline network system using the probability of system performance failure based on the supply damage rate as the index for evaluation is proposed, and an example seismic assessment is carried out for the actual water

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distribution network model considering the deterioration of pipelines.

2. Seismic Failure of the Existing Lifeline System

2.1 Analytical Model of Lifeline System

The water supply lifeline network system located in a metropolitan area of Japan is used as a sample model for this analysis. As shown in Fig. 1, the whole area is divided into several sub-zones which are the distribution districts to be controlled by the waterworks bureau of the city. The water lifeline network system has a hierarchical system composed of transmission lines, distribution and service network systems as shown in Fig. 1. The transmission pipelines have been constructed and maintained at a high seismic performance level, while the distribution and service networks partially include the old types of cast iron pipes (CIP) with mechanical joints as well as steel and ductile cast iron pipes (DCIP) with more seismically reliable joints. Due to this situation, potential seismic damages will be triggered from the weakest joints of cast iron pipes.

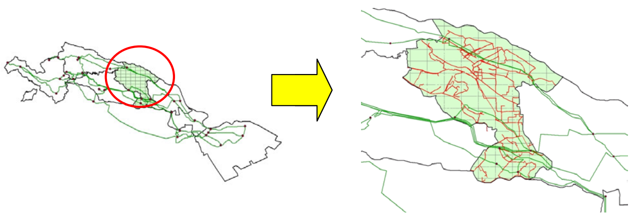


Figure 1. Water distribution network system for the assessment.

Table 1. Pipeline dimensions

	Pipe length (m)				
Dia.	CIP	DCIP		Steel Pipe	
mm	Old	Old	New	Old	New
200	4738	20091	0	1390	0
250	52	53	0	0	0
300	3620	14898	0	925	0
350	2820	14049	0	2121	0
400	0	0	0	879	0
500	0	7000	5260	0	23023
Subtotal	11230	56091	5260	5315	23023
Total	100919				

The main target of the seismic retrofitting activities should focus on replacing the old type of cast iron pipes with newly developed ductile cast iron pipes or arc-welded steel pipes of the distribution network system. Table 1 lists the pipe materials and their total lengths for each diameter.

2.2 Seismic Performance of Segmented Pipelines

(1) Failure mechanism of segmented pipelines

The old CIP and DCIP with the old type of joints are replaced with the new DCIP. Since the new DCIP has a locking system as shown in Fig. 2, it cannot easily be pulled out even in the case of large displacement by an earthquake. The seismic performances of these joints are compared in Table 2 as acceptable displacement in the axial direction.

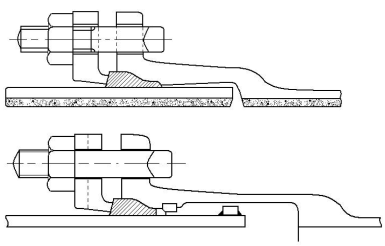


Figure 2. Old and new types of DCIP joints

Table 2. Seismic performance of each joint

Resistance of Joint (mm)

Type	Mean		
	Minor	Moderate	Major
Old joint	*5*	*10*	*15*
New joint	*30*	*45*	*60*

Type	COV		
	Minor	Moderate	Major
Old joint	*0.2*	*0.25*	*0.3*
New joint	*0.05*	*0.05*	*0.1*

The axial force S necessary to pull out from the joint is calculated with the shear stress which is sinusoidally distributed along the pipe axis. If the shear stress is less than the critical shear stress τ_{cr} that initiates slippage at the pipe surface, the maximum pull-out force is calculated with the shear stress τ_G given by the ground strain ε_G. When the shear stress is greater than the critical shear stress, on the other hand, the maximum pull-out force is derived from the critical shear stress given by Equation (1):

$$S = \begin{cases} \pi D \cdot l_{eff} \cdot \tau_G & \text{for} \quad \tau_G < \tau_{cr} \\ \pi D \cdot l_{eff} \cdot \tau_{cr} & \text{for} \quad \tau_G \geq \tau_{cr} \end{cases} \tag{1}$$

where,

l_{eff} : the effective length loaded by the shear stress acting on the pipe surface
τ_G : the shear stress acting between the pipe surface and the surrounding ground
τ_{cr} : the critical shear stress

Fig. 3 shows the shear strain distribution along the pipe stretching axis when the ground strain is larger than the critical shear strain ε_U necessary to initiate slippage at the pipe surface. In this figure, the structural shear strain distribution is enlarged from GCB to GFCB, because the excess strain of ABC (shaded area) is balanced by the additional strain of CFG.

As a simplified assumption, the following formula is used as a conservative estimation:

$$S = \pi D \cdot l_{eff} \cdot \tau_{cr} \tag{2}$$

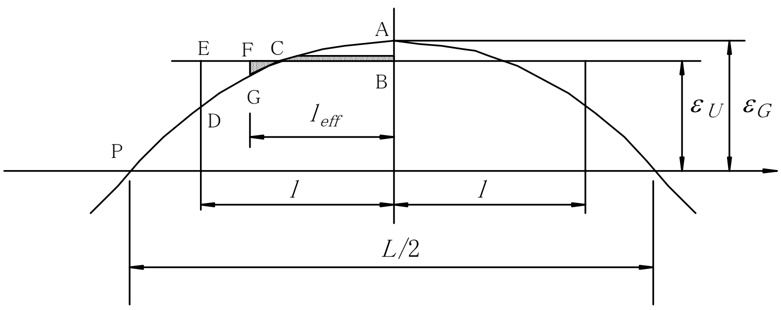


Figure 3. Ground and structural strains along the pipe stretching axis.

The resisting force for pulling out which is produced by the compression force at the contact surface between the locking ring and the stopper shown in Fig. 2 is given by the following equation:

$$R_Y = \pi D \cdot r \cdot \sigma_{cr} \tag{3}$$

in which

σ_{cr} : the critical axial stress to initiate failure by pull-out from the pipe joint
r: the height of the locking ring

The pull-out failure of the new DCIP initiates when the joint displacement $\Delta u_{G,eff}$ exceeds the critical pull-out displacement Δ_{joint}. Therefore, the pull-out failure can be assessed by the following criteria:

$$\Delta_{\text{joint}} < \Delta u_{G,eff} \quad : \quad \text{pull-out failure occurs.}$$
$$\Delta_{\text{joint}} > \Delta u_{G,eff} \quad : \quad \text{pull-out failure does not occur.} \qquad (4)$$

where the joint displacement is estimated as the relative displacement between the unit pipe length l_d shown in Fig. 4 in the following way:

$$\Delta u_{G,eff} = U_h \left\{ 1 - \cos\left(\frac{2\pi}{L} l_{eff} \right) \right\} \cong l_{eff} \cdot \varepsilon_G \qquad (5)$$

where,

L: wavelength

U_h: ground response displacement

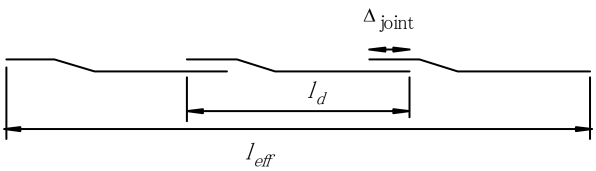


Figure 4. A group of mechanically jointed pipes moving together over the effective length.

(2) Effect of deterioration

The deterioration behavior of old pipe joints under traffic load vibrations is assumed to be modeled by the following deterioration factor:

$$\psi(t) = 1 - d_1 \left(\frac{t}{T_D} \right)^{d_2} \qquad (6)$$

where, d_1 and d_2 are parameters describing the deterioration behavior as shown in Table 3.

Table 3. Deterioration factors for pipe joints

	Deterioration factors	
Type	d_1	d_2
Old joint	0.5	0.25
New joint	0.1	0.05

Table 4. Seismic response at joint portion

Seismic Response (mm)		
Type	Mean	COV
EQ1	8	0.3
EQ2	37	0.4

In this assessment, the joint resistance with displacement in the pull-off direction represents the residual strength R. This value can be estimated based on the deterioration factor of Equation (6) as follows:

$$R(C_S, T_p) = \psi(T_p - T_0) \cdot R(C_S, T_0) \qquad (7)$$

(3) Fragility curves for segmented pipelines

Furthermore, seismic load S is evaluated as the relative displacement of the ground at the joint portion. Therefore, it can be calculated based on the seismic deformation method using the relative ground displacement at the next two joints:

$$\Delta u = U_h \left\{ 1 - \cos\left(\frac{2\pi \Delta l}{L} \right) \right\} \tag{8}$$

where, Δl is the distance between two joints. U_h is the horizontal displacement of ground[1] and is given by the following equation:

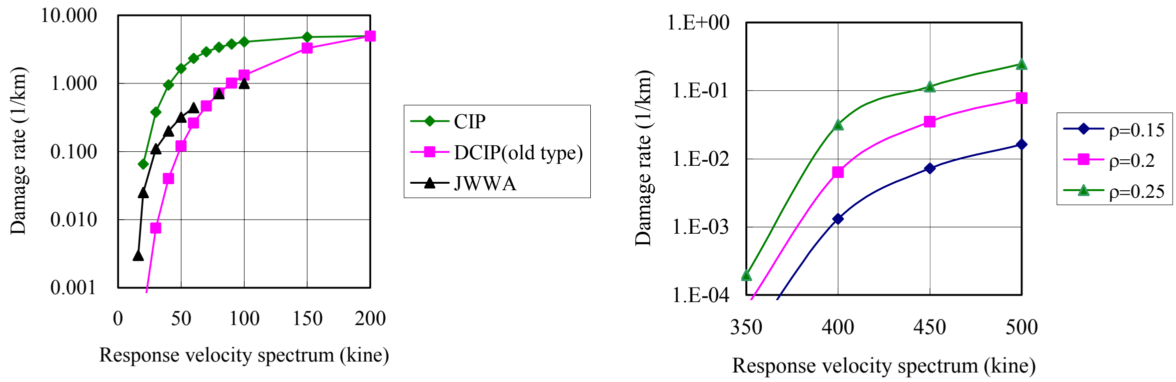
$$U_h = \frac{2}{\pi^2} S_V(T_G) \cdot T_G \tag{9}$$

where, T_G and S_V are the natural period of the ground surface and the ground response velocity at the natural period, respectively. In this assessment, T_G is simply assumed to be 1.0 second as the typical value over the entire area of the network system. The seismic responses of buried pipe joints for two types of seismic load are summarized in Table 4 in which the mean value and coefficient of variation (COV) of the axial displacement at the pipe joint are calculated from Equation (7).

Noting that potential damage points are identified for all mechanical joints at every unit length l_d, the number of potential damage points are given by $2l_{eff}/l_d$. Therefore, the damage rate, which is defined as the number of damages per kilometer, is estimated by:

$$\nu = E[\text{number of damages per wavelength}] \cdot \frac{1000}{L} = \frac{2l_{eff}}{l_d} \cdot P[R_Y < S_2 | EQ] \cdot \frac{1000}{L} \tag{10}$$

Using Equation (10), the damage rates for CIP and DCIP (old type joint and new type joint) are estimated for various response velocities at the ground surface as shown in Figure 5. Especially, Figure 5 (1) compares the damage rate for old type joints of CIP and DCIP, and that for JWWA (Japan Water Works Association) formula which is given by the empirical curve based on the damage data of water pipelines in 1995 Kobe Earthquake.



(1) Old-type joints

(2) New-type joints

Figure 5. Damage rates for CIP and DCIP.

The damage rate of JWWA is located between the damage rates of CIP and DCIP (old type), which suggests the applicability of the proposed method. Figure 5 (2), on the other hand, shows the damage rates for DCIP (new type joint) with three different connection factors r which is a factor to control the number of mechanically locked joints along the effective length l_{eff} in the seismic wave propagation in Figure 3.

3. Seismic Risk Assessment

The required seismic performance of a lifeline system can be defined as shown in Table 5. Both the seismic performance damage mode and the component damage mode are also defined quantitatively for seismic disasters of Level 1 (EQ1) and Level 2 (EQ2) ground motions caused by the maximum operational earthquake (MOE) and the maximum credible earthquake (MCE) respectively.

Table 5. Definition of seismic performance and failure modes

Seismic performance	Definition	Performance damage mode	Component damage mode
1	The system performance can be maintained without any disruption for the Level 1 earthquake ground motion (EQ1), if the system is slightly damaged or undamaged	minor D_{EQ}^{minor}	minor Z^{minor}
2	The system performance can be restored after quick repair for the Level 2 earthquake ground motion (EQ2), if the system is not significantly damaged	moderate $D_{EQ}^{moderate}$	moderate $Z^{moderate}$
3	The system performance can be restored after recovering from disruption for the Level 2 earthquake ground motion (EQ2), if the system is not completely damaged	major D_{EQ}^{major}	major Z^{major}

3.1 Structural Failure

Let us define the damage modes for structural components in the following way, where *R*, *D*, *L*, *S*, C_S, T_p are the residual strength for each damage mode, dead load, live load, seismic load, retrofitting investment and present time, respectively:

$$\begin{aligned} Z_{EQ}^{\text{minor}}(C_S,T_p) &= R^{\text{minor}}(C_S,T_p)-(D+L)-S\cdot 1_{EQ} \\ Z_{EQ}^{\text{moderate}}(C_S,T_p) &= R^{\text{moderate}}(C_S,T_p)-(D+L)-S\cdot 1_{EQ} \\ Z_{EQ}^{major}(C_S,T_p) &= R^{major}(C_S,T_p)-(D+L)-S\cdot 1_{EQ} \end{aligned} \tag{11}$$

where,

$$1_{EQ}(t_{EQ}) = \begin{cases} 1: \text{an earthquake occurs at } t_{EQ} \\ 0: \text{an earthquake does not occur at } t_{EQ} \end{cases} \tag{12}$$

The expected value of 1_{EQ} equals the probability of an earthquake occurring in the residual service period $(T_D\text{-}T_p)$ and it is given by the following equation using the return period of the possible future earthquake:

$$E\left[1_{EQ}\left(t_{EQ}\right)\right]=P[EQ]=1-\left(1-\frac{1}{T_R}\right)^{T_D-T_p} \tag{13}$$

Then, the corresponding probabilities of component damage mode are denoted as:

$$p_{fi}^{B}=P\left[Z_{EQ}^{\min or}<0\right]\ ,\ \ p_{fo}^{B}=P\left[Z_{EQ}^{\mathrm{mod}\, erate}<0\right],\ \ \ p_{fa}^{B}=P\left[Z_{EQ}^{major}<0\right] \tag{14}$$

Assuming that damage in a pipeline follows a Poisson process, the probability of pipe failure[2] in the *i*-th link is given by:

$$\begin{aligned} P\left[Z_{link}^{\text{minor}}(l_i)<0\middle|EQ_1\right]&=1-\exp\left[-\int_0^{l_i}\nu_{\text{minor}}(x)dx\right] \\ P\left[Z_{link}^{\text{moderate}}(l_i)<0\middle|EQ_2\right]&=1-\exp\left[-\int_0^{l_i}\nu_{\text{moderate}}(x)dx\right] \\ P\left[Z_{link}^{\text{major}}(l_i)<0\middle|EQ_2\right]&=1-\exp\left[-\int_0^{l_i}\nu_{\text{major}}(x)dx\right] \end{aligned} \tag{15}$$

where the damage rate per unit pipe length at point x is given by:

$$\nu_{mz}(x)=\nu_0 P\left[Z_{EQ}^{mz}\middle|EQ\right] \tag{16}$$

where, *mz* indicates the structural damage mode of minor, moderate and major, and ν_0 denotes the number of pipe joints per unit pipe length.

3.2 Performance Failure

The number of failures in the *i*-th link corresponding to structural damage mode *mz* is given by the following equation:

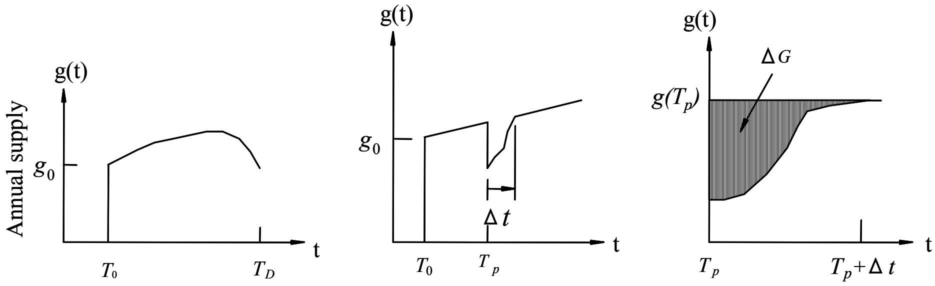
$$n_i^{mz}=\int_0^{l_i}\nu_{mz}(x)dx \tag{17}$$

The performance level of the network system after the seismic event can be estimated by the supply damage rate (ratio of unserviceable users to all users of the water supply system). Kawakami[3] suggested a simple method to estimate the supply damage rate of water after a seismic event from the structural damage rate of the water distribution network system. In this study, the supply damage rate, which is defined for the structural damage mode of *mz* and the system performance failure mode of *md*, is assumed to be a function of the damage occurrence rate $\nu_{mz}(x)$ in Equation (18):

$$\gamma_{mz}(t,x)=\frac{1}{1+a\{\nu_{mz}(x)\}^b} \tag{18}$$

where Kawakami suggested that $a = 0.0473$ and $b = -1.61$.

Fig. 6 shows a conceptual illustration of a short disruption of lifeline service[4] during the restoration period after seismic damage has occurred. Fig. 6 (1) shows a profile of water serviceability in the life cycle stage between the initial time point T_0 and the terminal time point T_D. When an earthquake occurs at T_p, some restoration time Δt is necessary after the quake as shown in Fig. 6 (2). The detailed restoration process is shown in Fig. 6 (3), in which ΔG is the loss of serviceability caused by seismic damage.



(1) Profile in the life cycle period (2) Quake occurrence (3) Restoration process

Figure 6. Schematic profile of water serviceability in the life cycle stage and malfunction during the restoration process after the earthquake.

The restoration process is shown in more detail in Fig. 7, in which the process is divided into four time points of T_1, T_2, T_3 and T_4. The time point T_1 is the next day after the earthquake where the water serviceability level is provided to non-damaged areas which are identified by a site survey of damage points along all the pipelines. The time point T_2 is when the transmission mains are recovered, while the time point T_3 is the time when temporary pipelines are installed for emergency use in the damaged areas. T_4 is the completion time when all the damaged pipelines are restored.

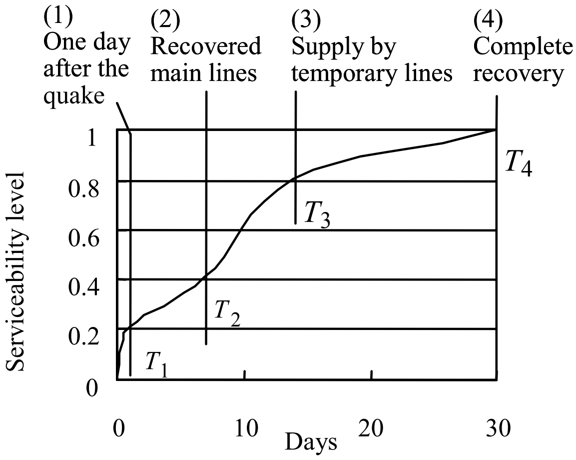


Figure 7. Schematic profile of restoration process after the earthquake.

The water supply level at these four time points in the restoration process of the water distribution area x is denoted as the water supply damage rate as follows:

$$\gamma_k(x) = \gamma_{major}(T_k, x) \quad \text{for } k = 1,2,3,4 \tag{19}$$

in which γ_1 is related with the system connectivity between the supply nodes and demand nodes while γ_2 is obtained by flow analysis of the damaged network. When temporary lines are completed at T_3, the target serviceability level is expressed as γ_3. The water supply level over the distribution areas is summarized by the following equation:

$$\Gamma_{major}(T_p) = \frac{\int_0^{T_4} \int_A \gamma_{major}(t, x) a(T_p, x) dx dt}{\int_A a(T_p, x) dx} \tag{20}$$

Therefore, the supply damage rate in the damaged network system can be expressed by the following Equation (21):

$$P\left[D_{EQ}^{major} \middle| \bigcup_{i=1}^{NL} \{ Z_{link}^{mz}(l_i) < 0 \} \right] = \Gamma_{major}(T_p) \tag{21}$$

where, D_{EQ}^{major} is the major damage state in terms of serviceability of water supply.

4. Seismic Mitigation

4.1 Conditions of Numerical Studies

Numerical simulations are carried out for several alternatives to obtain the optimal seismic mitigation strategy. Fig. 8 shows a schematic diagram of these investment schemes, in which C_S, T_0, T_p, T_D, J_R are the seismic investment, the start point of supply service, the present point, the point of service end, and the ratio of old-type joint pipes to all pipes, respectively. Scheme 1 is an intensive retrofitting plan, while scheme 3 is a long-term plan, and scheme 2 is an intermediate plan. Numerical conditions are also summarized in Table 6.

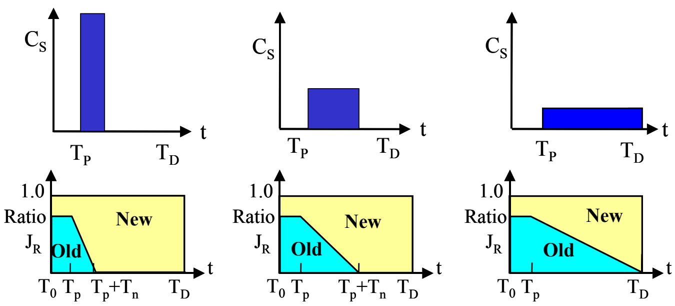


Figure 8. Assumed seismic investment schemes.

Table 6. Numerical conditions

Item	Symbol	Unit	Amount
Total pipe length	L	*km*	*100*
Unit length of pipe	u_L	*m*	*5*
Restoration period	Δt	*month*	*0.5*
Return period of EQ1	T_{R1}	*year*	*50*
Return period of EQ2	T_{R2}	*year*	*475*
Retrofitting period	T_n	*year*	*10,30,70*

4.2 Simulation Results

Fig. 9 shows the probability of system performance failure for various retrofitting periods under the seismic investment ratio $C_S/C_0 = 0.5$, in which C_S and C_0 are the seismic investment cost and initial cost, respectively. As shown in Fig. 10, all existing old pipes are replaced when C_S/C_0 exceeds 0.4. Therefore, this is the case when sufficient seismic investment is prepared. Rapid retrofitting cases indicate that the improvement in seismic performance is remarkable in the minor damage mode. This result shows that the intensive retrofitting plan is effective to reduce the seismic risks when sufficient investment is prepared.

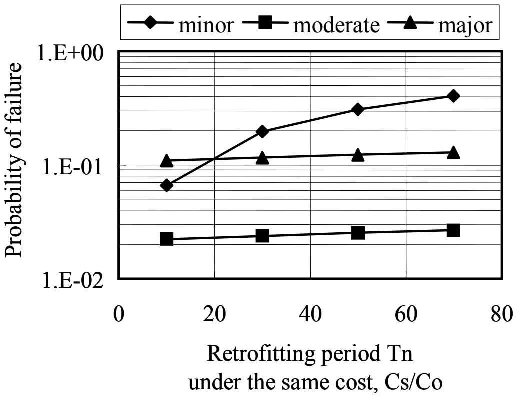


Figure 9. Probability of system failure for various retrofitting periods under $C_S/C_0 = 0.5$.

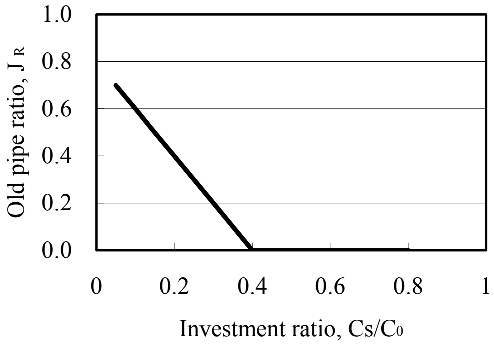


Figure 10. Old pipe replacement for various seismic investments.

Fig. 11 shows the probability of system performance failure for various C_S/C_0 when the retrofitting period T_n is 30 years. The seismic performance is clearly changed at $C_S/C_0 = 0.4$. Especially, this probability in the minor damage mode shows a sudden drop. This result indicates that the existence of old-type joint pipes affects the seismic performance of the deteriorated network system.

Fig. 12 indicates the effects of deterioration on seismic retrofitting cost by year of occurrence of a seismic event. Since the severe deterioration case shows a higher retrofitting cost, the durability of the pipe material is also an important factor for the seismic performance of the lifeline network.

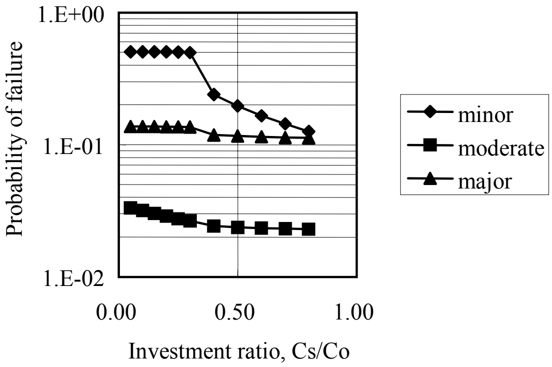


Figure 11. Probability of system failure for various seismic investments.

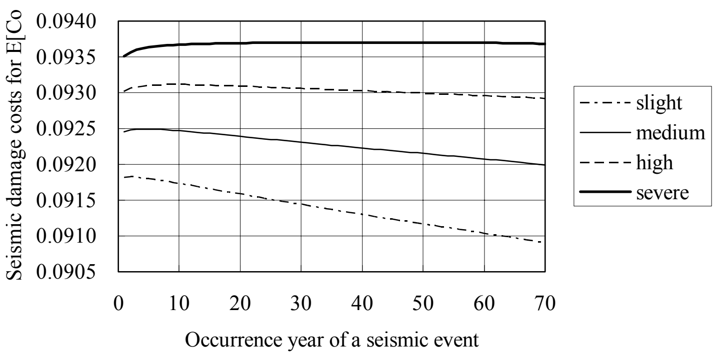


Figure 12. Effect of deterioration on seismic retrofitting cost

5. Conclusions

In this study, an assessment procedure using the probability of system performance failure for the existing lifeline network system was proposed and case studies were carried out in order to evaluate the effectiveness of retrofitting strategies for the existing water distribution network model.

Several important results can be summarized as follows:

(1) The probability of system performance failure is related to the probability of structural failure through the water supply damage rate.

(2) The failure modes of deteriorated pipelines can be modeled based on the performance-based design method and their fragility curves can be obtained.

(3) Effective planning of seismic retrofitting activities and disaster mitigation for the existing lifeline network system can be achieved using the newly proposed assessment method.

References

[1] JWWA, 1997. *Seismic design guideline for waterworks pipeline and facilities,* Japan Water Works Association (JWWA).

[2] Mochizuki, F. & Koike, T. 2007. *Performance of water lifeline network with damaged node structures under seismic risk.*, Int. Conf. on Applications of Statistic and Probability in Civil Eng., Tokyo.

[3] Kawakami, E. 1984. *Relationship between damage ratio and normal-supply ratio for simple lifeline system.*, J. of Japan Society of Civil Engineers, No.344/I-1

[4] Imai, T. & Koike, T. 2009. *Seismic risk management for an existing lifeline system*, ICOSSAR2009, Osaka., Japan.