2.0 of the ration. Since the correction factor is different in microtopograpy type in the prediction method, the relation between the actual number of damage and predicted one in each microtopograpy type is drawn in Figures 2 and 3. The actual number of damage is relatively lager than the predict one in case of law correction factors for microtopograpy type according to Figure 2. This discrepancy is probably due to land use change for housing development. Thus, the microtopograpy map of 250m x 250m cell is not sufficient for locally change of landform.

Figure 4 is illustrated the relation between an actual number of damage and predicted one in liquefaction area in Urayasu. The correction factor is 6.0 in spite of microtopograpy type in the cell in this case. Most of data are plotted at the area above 2.0 of the ration of the actual number of damage to the predicted one. This means that the predicted number of damage is underestimated very much.

The total piping length in Urayasu was 200.93 km and the number of damage was 321, so the damage rate is 1.60 locations/km. The damage rate of pipeline buried in the reclaimed land of Kobe, Ashiya and Nishinomiya in the 1995 Kobe Earthquake was 1.77 locations/km. Peak ground velocity was about 30 cm/s in Urayasu and that in Kobe area in the 1995 Kobe Earthquake was around 100 cm/s. So, peak ground velocity of Urayasu was much smaller than that at Kobe area in the 1995 Kobe Earthquake, but degree of damage was seems to be similar. This means that the effect of liquefaction on pipeline damage may be nearly constant, once it had occurred, and independent of peak ground velocity. Therefore, some modification is necessary in the prediction method in case of liquefaction.

Table 1 Pipeline damage prediction equation

$Rm(v) = C_p \times C_d \times C_g \times R(v)$ $Rm(v)$: Predicted damage rate [locations/km] $R(v)$: Reference damage rate [locations/km] $= 9.92 \times 10^{-3} \times (v - 15)^{1.14}$ v: Peak ground velocity (cm/s) $(15 \leq v < 120)$			
Pipe type, joint	C_p	Diamater(mm)	C_d
DIP (A)	1.0	ø50–80	2.0
DIP (K)	0.5	ø100–150	1.0
DIP (T)	0.8	ø200–250	0.4
DIP (disengagement prevention)	0	ø300–450	0.2
CIP	2.5	ø500–900	0.1
PVC (TS)	2.5		
PVC (RR)	0.8		
SP (welding)	0.5/0		
SP (non-welding)	2.5		
ACP	7.5		
PE (electrofusion)	N/A※		
Microtopography with pipelines installed			C_g
If there is no information available on liquefaction or if there is no possibility of liquefaction			
Mountain, mountain foot, hill, volcanic area, volcanic mountain foot, volcanic hill			0.4
Gravel upland, loam upland			0.8
Valley floor, alluvial fan, backswamp, delta, coastal lowland			1.0
Natural levee, former river channel, sandbar, gravel bar, dune			2.5
Reclaimed land, drained land, lakes and marshes			5.0
If there is information available on liquefaction and if there is the possibility of liquefaction			
All topography types			6.0

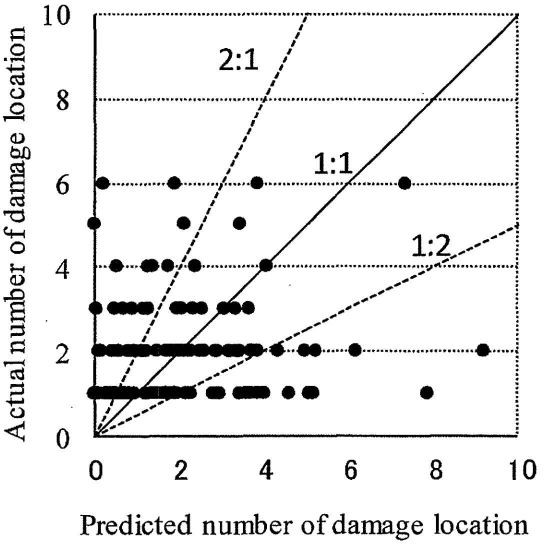

Figure 1 Comparison of actual number of damage with predicted one in Ishinomaki.

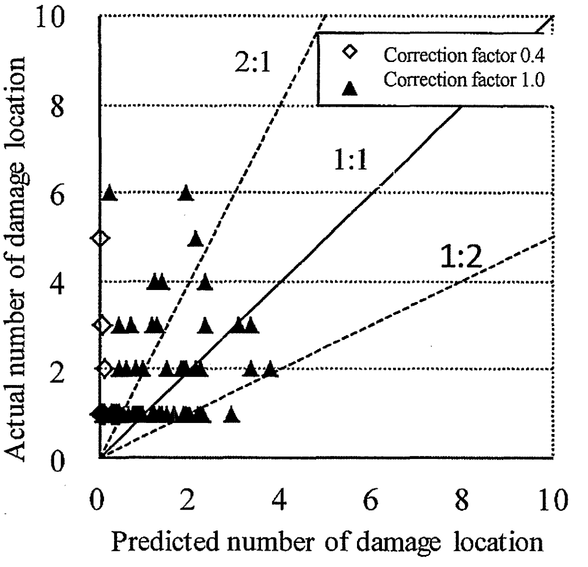

Figure 2 Comparison of actual number of damage with predicted one of correction factors 0.4 and 1.0.

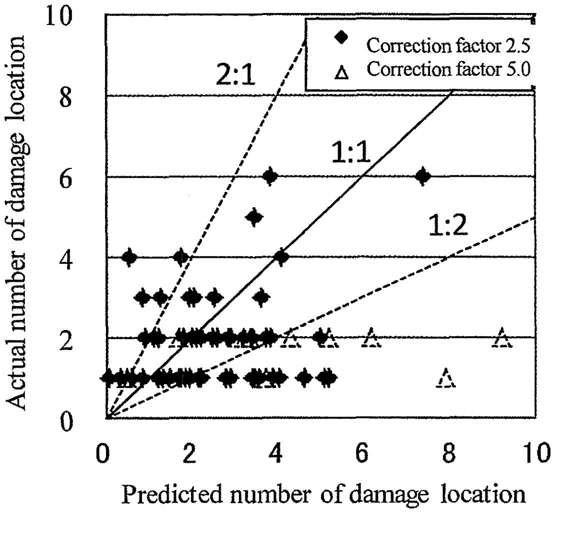

Figure 3 Comparison of actual number of damage with predicted one of correction factors 2.5 and 5.0.

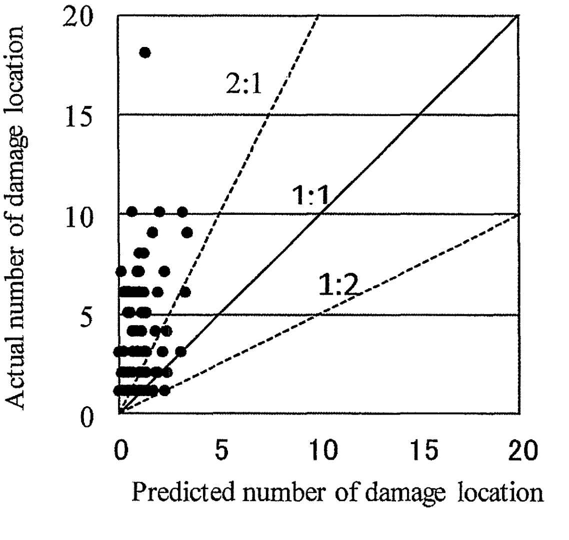

Figure 4 Comparison of actual number of damage with predicted one in Urayasu.

Estimation of Correction Factor in Liquefaction Area

If liquefaction occurred, the effect of liquefaction on pipeline damage seems to be constant and independent of peak ground velocity. An average damage rate in liquefaction area must be estimated independently of peak ground velocity. The average damage rate of liquefaction area is estimated by reference damage rate divided by correction factors on pipe type and pipe diameter by using database of Urayasu and Asahi in the 2011 Great East Japan Earthquake. The average damage rate of liquefaction area is finally set to 5.5.

Figure 5 shows the relation between an actual number of damage and predicted one in liquefaction area by using the average damage rate of liquefaction area, in comparison with that using the damage prediction equation in Urayasu. 60.4 % of all data are plotted between two dotted lines and only 14.3% are plotted above 2.0 of the ratio. So, comparison of the predicted number of damage with the actual one shows good agreement.

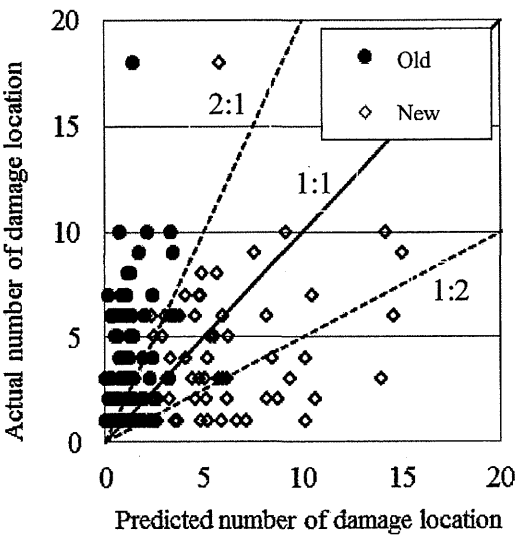

Figure 5 Comparison of actual number of damage with predicted one by using the average damage rate of liquefaction area in Urayasu.

Finally the method of damage prediction of water supply pipeline is newly proposed as shown in Table 2.

Table 2 Newly proposed pipeline damage prediction equations

Pipeline damage prediction equations	
If there is no information available on liquefaction or if there is no possibility of liquefaction	If there is information available on liquefaction and if ther is the possibility of liquefaction
$R_m = C_p \times C_d \times C_g \times R(v)$ R_m : Predicted damage rate (locations/km) C_p : Correction factor for pipe and joint type C_d : Correction factor for pipe diameter C_g : Correction factor for microtopograpy $R(v)$: Reference damage rate (locations/km) $R(v)=9.92\times10^{-3}\times(v-15)^{1.14}$ v : Peak ground velocity(cm/s) $(15 \leqq v < 120)$	$R_m = C_p \times C_d \times R_L$ R_m : Predicted damage rate (locations/km) C_p : Correction factor for pipe and joint type C_d : Correction factor for pipe diameter R_L : Average damage rate of liquefaction area (locations/km), $R_L = 5.5$

C_p, C_d, C_g are refered to Table 1.

CONCLUSION

The equation for prediction of earthquake damage to water supply pipelineproposed by authors is verified by using damage data in the 2011 Great East japan Earthquake. Since comparison between the damage predicted by using the present equation and the actual damage occurred at strong ground motion areas in the 2011 Great East Japan Earthquake shows good agreement, the present method is verified. It is, however, clarified that the microtopograpy map of 250m x 250m cell is not sufficient for locally change of landform. The damage to pipeline in liquefied areas is, however, not explained precisely by using the present prediction method because liquefaction occurred extensively at the area of relatively low peak ground velocity such as Urayasu. If liquefaction occurred, the effect of liquefaction on pipeline damage seems to be constant and independent of peak ground velocity. Modified prediction method of damage to pipeline in liquefied area is, therefore, proposed based on the database of the damage in the 2011 Great East Japan Earthquake in this paper.

ACKNOWLEDGMENTS

This study was supported in part by the Grant in Aid for Science Research from the Ministry of Health, Labor and Welfare, Japan. The authors would like to gratefully acknowledge the working group members consisting of engineers from water utilities/companies for their technical support.

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Application of Two-stage Coagulation for High Turbidity Raw water

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As a result of frequent occurrence of heavy rain, widely applicable water treatment processes for high turbidity raw water was required. In the presence of high turbidity, traditional coagulation processes can be inadequate to water quality standards such as *Cryptosporidiidae*. Two-stage coagulant addition easily applies many existing water purification plant and expects to treat high turbidity raw water effectively. In this study, high turbidity raw water was used to compare one- and two-stage coagulant addition on conventional water treatment process. Jar test were conducted with various water quality conditions and verified at the pilot scale experiment. The settled-water turbidity and particle counts were approximately 20% and 50% lower than for the single-stage process less using the two-stage process respectively. Though, the steady-state filter effluent turbidity for the two-stage process was approximately 20% lower than for the single-stage process, the one-stage process did not exhibit particle breakthrough during a typical filter run, whereas the two-stage process broke through approximately 4hr before the hydraulic completion of the run. Additional experiments allowed a mechanism for the improved process performance to be proposed. As a simple and efficient approach, two-stage coagulation could have great practical significance in conventional water treatment process.

Simplified Evaluation Method
of Seismic Resistance for Water Treatment Facilities

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ABSTRACT

As the Seismic Design Code (SDC) has revised its requirement following major earthquakes in the past decades, old water infrastructure in Japan must be seismically upgraded to keep up with the latest SDC.
As a first step, such upgrade usually requires an assessment whether a specific civil structure is seismically resistant. The Detailed Evaluation Method (DEM) is well-known as a full evaluation method, but due to its complexity and required high technical knowledge, the Simplified Evaluation Method (SEM) is normally applied to prioritize the structures for DEM testing. This simple assessment method, however, falls behind the current civil engineering technology in some respects, being thirty years old.
Therefore, we propose a new SEM with a revised procedure, which adopts a three-step approach where the target structure is evaluated according to "threat of liquefaction," "year of construction" and then "structural resistibility."

INTRODUCTION

Japan is located in one of the most active seismic zones in the world, and undertaking adequate seismic countermeasures is crucially important to protect the public water supply from seismic hazards or restore it promptly after such an event. This is why Japan's drinking water infrastructure is designed in compliance with the Seismic Design Code (SDC). The first edition of the SDC was defined in the *Guidelines for Earthquake-Resistant Design Methods for Waterworks Facilities*, which was published in 1953 in the wake of the 1948 Fukui Earthquake. Since then the SDC has undergone several revisions to adopt more rigid criteria, e.g., after the 1964 Niigata Earthquake and most recently after the 1995 Kobe Earthquake. Installed before or in the 1960s-1970s, a considerable proportion of the water infrastructure remains to be seismically upgraded and meet the latest SDC.

As a first step to an upgrade, a drinking water utility usually makes an assessment whether a specific civil engineering structure at a water treatment plant (WTP) is seismically vulnerable and must be reinforced. The Detailed Evaluation Method (DEM) is available as a common, full evaluation method, in which the target structure is evaluated through the simulated calculations of the stress imposed on each structural member under specific seismic intensities and motions as well as other design conditions. However, as the DEM requires high technical knowledge as well as time and effort, the Simplified Evaluation Method (SEM) is normally applied as a preliminary step to identify and prioritize which structures need a thorough evaluation by the DEM. As a qualitative assessment method, the SEM adopts a scorecard (Diagnostic Sheet) approach where the user grades the target structure for various indices to know the level of its seismic resistance. Established in 1981 by the Ministry of Health and Welfare, however, the SEM is partially outdated, not necessarily reflecting the current civil engineering technology.

With the aim of revising the SEM and its Diagnostic Sheet to make them more practical and convenient to use, the Japan Water Research Center (JWRC) started a three-year research project in 2011 in collaboration with utilities and academics under Health Labour Sciences Research Grant.

This paper discusses the current progress and findings of the ongoing project.

METHOD

We collect the DEM results from utilities and apply the SEM to the same structures tested by the DEM in order to compare the DEM and SEM results and identify weaknesses of the Diagnostic Sheet. We also investigate the damage to the facilities by the Great East Japan Earthquake in March 2011 to reflect the actual damage observations in revising the SEM.

Survey of damage caused by the Great East Japan Earthquake. To find out the damage to the facilities by the Great East Japan Earthquake, we sent out a questionnaire to the five government agencies in five affected prefectures (Iwate, Miyagi, Fukushima, Ibaraki, and Chiba). These agencies regulate the utilities in respective prefectures and receive the damage reports following earthquake events. The questionnaire did not address the damage related to the inundation due to tsunami. Upon receiving responses, we conducted on-site investigations of the facilities reported to have sustained particularly severe damage.

Proposals to improve the SEM. As a case study we applied the Diagnostic Sheet approach to two groups of structures: the one that was designed according to older SDCs and the other conforming to the latest SDC. We examined improvement measures for the SEM based on the case study results as well as the reports on the damage in the Great East Japan Earthquake and other earthquakes.

RESULTS AND DISCUSSION

Damage by the Great East Japan Earthquake. Of the five government agencies, four responded to the questionnaire. The result showed there were breakage or fall of plate settlers at some WTPs but there were none but three plants that were forced to suspend water supply for extended periods due to critical damage. Subsequently we made on-site investigations of the three plants (Hebita in Miyagi, Wanigawa in Ibaraki, and Shinjuku in Chiba). The follow-up investigations revealed that the primary cause of the damage was not the ground shaking itself but the ground failure due to liquefaction. Afterwards we examined and found this was also the case in the 1995 Kobe Earthquake as well as subsequent earthquakes: the critical impact on the WTPs was brought about largely by the liquefaction effect, and the other effects played a minor role.

Photo 1. Land subsidence due to liquefaction (Shinjuku WTP)

Photo 2. Concrete expansion joint forced widened due to ground deformation (Hebita WTP)

Weakness of the current SEM. The "total score" approach through the Diagnostic Sheet (Table 1) is one of the most common among various SEMs currently available. In this approach, the user grades the target structure in numeric values (scores) for a series of indices from "Ground Condition" to "Seismic Intensity" in Table 1 and multiplication of all the indices leads to Seismic Resistibility. This is a common seismic vulnerability assessment but is not without some weaknesses: for example, the user must fill in every index and to grade every index is not always easy to smaller utilities due to limited personnel and financial resources. Furthermore, developed in 1981, this approach is not designed based on the latest seismic intensity scale defined by the Japan Meteorological Agency in 1996. Addressing these weaknesses in a proper fashion is a motivation behind our research project.

Table 1. SEM Diagnostic Sheet

Type of Structure	Roofed Basin (Distribution reservoir, Clear water tank)			
Name of Facility	A Distribution Reservoir			
Item	Category	Weight	Score	Remarks
Ground Stiffness	Type I	0.5		
	Type II	1.5		
	Type III	1.8		
Liquefaction	No	1.0		
	Possible	2.0		
	Yes	3.0		
Ground Formation	Natural ground	1.0		
	Slope	1.2		
	Hilltop	1.3		
	Embankment/landfill	1.5		
Basement Level	Onground	1.2		
	Semi inground	1.1		
	Inground	1.0		
Material	RC	1.0		
	Bricks/others	3.0		
Wall Area/ Water Area*	0.05<	1.0		
	0.05>	1.5		
Depth of Reservoir	5m≧	1.0		
	5m<	1.3		
Modeling Type for Structural Analysis	Wall	1.0		
	Pillar & baem	1.2		
	Flat slab	1.4		
Upper Load	0.4m≧	1.0		
	0.4m<	1.2		
Construction Year**	Befor 1953	1.8		
	1953～1966	1.6		
	1967～1980	1.5		
	After 1980	1.0		
Flexible Joint (connected pipe)	Mounted	1.0		
	Nil	2.0		
Expansion Joint (wall, slab, etc.)	Good condition	1.0		
	Nil or no good	2.0		
Aging	Relatively new	1.0		
	Medium	1.5		
	Aged	2.0		
Seismic Intensity	5	1.0		
	6	2.2		
	7	3.6		
Seismic Resistibility (score)	High	10>		
	Middle	10～17		
	Low	17<		

* Wall Area/Water Area = [horizontal cross-sectional area of wall] / [water surface area]
**Category in "Construction Year" was amended in 2000 by JWRC

Procedure of the new and Revised SEM. The Diagnostic Sheet approach needs to be simplified if an increased number of smaller utilities are to be encouraged to undertake more appropriate seismic countermeasures. Based on the two observations (1) grading every index can be difficult in some cases, and (2) that the critical damage was the result of the liquefaction effect, we propose the Revised SEM procedure as follows (Figure 1):

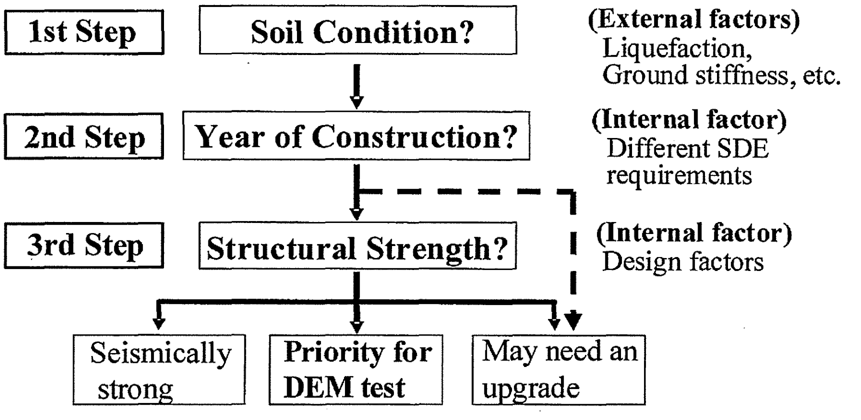

Figure 1. Procedure of Revised SEM

First step: Evaluate the target structure according to external factors. External factors include ground stiffness, possibility of liquefaction, and ground formation. This evaluation is reasonable as these factors seem to account for far more cases of damage than other factors.

Second step: Evaluate the target structure by the year of construction. As we have mentioned, structures must be designed in compliance with the latest SDC available at the time, and the SCD's requirements vary depending on its edition. The year of construction helps to identify the levels of seismic design adopted by the target.

The first and second step can be done in the reversed order or simultaneously.

Third step: Assessment of structural load-bearing capacity. For the structures that passed the first and second steps, assess their structural resistibility against seismic motion, that is, their structural load-bearing capacity.

Considerations on structural load-bearing capacity. To find issues of the current Diagnostic Sheet in assessing the structural resistibility against seismic motion, and to examine the appropriateness of the indices adopted in the Sheet, we applied the current SEM to two groups of rectangular clear-water tanks/distribution reservoirs (reservoirs) in several water utilities.

The first group of reservoirs (Group 1) is shown in Table 2. Group 1 was designed based on older SDCs and, as a result of DEM by the respective utilities, assessed to be seismically vulnerable. The DEM result indicated that major defective structural members were the side walls and pillars, with their edges especially vulnerable as the stress concentrates due to ground shaking.

Table 2. Reservoirs conforming to older SDCs (Group 1)

Facility	Construction Year	Structure Type	Capacity	Water Depth	Foundation Type
Reservoir A	1980	RC Flat slab (inground)	7,200 m^3	4.5m	Piled
Reservoir B	1989	RC Flat slab (inground)	5,000 m^3	4.5m	Spread
Reservoir C	1977	RC Flat slab (inground)	500 m^3	4.3m	Spread
Reservoir D	1976	RC Flat slab (inground)	300 m^3	3.2m	Spread
Reservoir E	1962	RC Wall (inground)	700 m^3	3.5m	Spread
Reservoir F	1977	RC Wall (semi inground)	330 m^3	3.5m	Spread
Reservoir G	1980	RC Wall (inground)	200 m^3	2.7m	Spread

Table 3 shows the second group of reservoirs (Group 2). Group 2 conforms to the latest SDC and thus considered seismic proof. For this reason this Group had not been DEM-tested.

Table 3. Reservoirs conforming to the latest SDC (Group 2)

Facility	Construction Year	Structure Type	Capacity	Water Depth	Foundation Type
Reservoir H	2011	RC Flat slab (inground)	32,600 m^3	6.4m	Spread
Reservoir I	1999	RC Flat slab (inground)	24,000 m^3	5.3m	Spread
Reservoir J	1999	RC Flat slab (semi inground)	4,500 m^3	4.4m	Spread
Reservoir K	2000	RC Flat slab (semi inground)	8,500m^3	3.5m	Spread
Reservoir L	2008	RC Flat slab (semi inground)	700m^3	4.0m	Spread
Reservoir M	2000	RC Flat slab (semi inground)	1,000m^3	5.0m	Spread
Reservoir N	2007	RC Flat slab (semi inground)	130m^3	5.0m	Spread

We applied the current SEM to Group 1 and Group 2. As shown in Table 4, Group 1 reveals the seismic resistance of either Low or Middle, while Group 2 marks Middle or High. Although some reservoirs in Group 1 get better points than ones in Group 2, the result seems relatively reasonable.

Table 4. Result of Diagnostic Sheet assessment

	Group 1 (Seismically Vulnerable)							Group 2 (Seismic Proof)						
	A	B	C	D	E	F	G	H	I	J	K	L	M	N
Ground Stiffness	1.5	1.5	1.8	0.5	0.5	0.5	0.5	1.5	1.5	0.5	0.5	0.5	0.5	0.5
Liquefaction	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Ground Formation	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Basement Level	1.0	1.0	1.0	1.0	1.0	1.1	1.1	1.0	1.0	1.1	1.1	1.1	1.1	1.2
Material	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Wall Area/Water Area	1.5	1.5	1.0	1.5	1.0	1.5	1.5	1.5	1.5	1.5	1.5	1.0	1.0	1.0
Depth of Reservoir	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.3	1.3	1.3	1.0	1.0	1.3	1.3
Modeling Type	1.4	1.4	1.4	1.4	1.4	1.0	1.0	1.4	1.4	1.4	1.4	1.4	1.4	1.0
Upper Load	1.0	1.2	1.0	1.0	1.0	1.0	1.0	1.0	1.2	1.0	1.2	1.0	1.2	1.0
Construction Year	1.5	1.0	1.5	1.5	1.5	1.5	1.5	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Flexible Joint (pipe)	2.0	2.0	2.0	2.0	2.0	2.0	2.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Expansion Joint (wall)	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Aging	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Seismic Intensity	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
Score	**51.0**	**40.8**	**40.8**	**17.0**	**11.3**	**13.4**	**13.4**	**14.7**	**17.7**	**5.4**	**5.0**	**2.8**	**4.3**	**2.8**
Seismic Resistibility	Low	Low	Low	Low	Mid.	Mid.	Mid.	Mid.	Mid.	High	High	High	High	High

On the other hand, Table 5, which is an extract from Table 4, shows the indices related to the structural resistibility. On average, Group 2 marks higher scores (meaning seismically weaker) than Group 1, and this symptom is obvious in some of the shaded reservoirs in Table 5.

Table 5. Result of Diagnostic Sheet assessment for structural resistibility (extract)

	Group 1 (Seismically Vulnerable)							Group 2 (Seismic Proof)						
	A	B	C	D	E	F	G	H	I	J	K	L	M	N
Basement Level	1.0	1.0	1.0	1.0	1.0	1.1	1.1	1.0	1.0	1.1	1.1	1.1	1.1	1.2
Material	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Wall Area/Water Area	1.5	1.5	1.0	1.5	1.0	1.5	1.5	1.5	1.5	1.5	1.5	1.0	1.0	1.0
Depth of Reservoir	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.3	1.3	1.3	1.0	1.0	1.3	1.3
Modeling Type	1.4	1.4	1.4	1.4	1.4	1.0	1.0	1.4	1.4	1.4	1.4	1.4	1.4	1.0
Upper Load	1.0	1.2	1.0	1.0	1.0	1.0	1.0	1.0	1.2	1.0	1.2	1.0	1.2	1.0
Score	**2.1**	**2.5**	**1.4**	**2.1**	**1.4**	**1.7**	**1.7**	**2.7**	**3.3**	**3.0**	**2.8**	**1.5**	**2.4**	**1.6**

While Group 2 showed high scores in the overall assessment result (Table 4), it did not perform as well in the structural resistibility analysis (Table 5).
The current structural analysis works in disfavor of a structure with a wider wall and/or area. The water area of a larger reservoir naturally becomes wider, but this translates into a lower assessment of "Wall area/Water area," and thus into a lower structural resistibility assessment, whereas the same structure may be assessed "seismic proof" in the overall assessment. The main cause of the problem is: whereas five indices in Table 5 (basement level, material, reservoir depth, modeling type, upper load) only represent design conditions having little impact on the structural resistibility assessment, "Wall area/Water area" influences the assessment disproportionately as the single most important index.
Judging from the above, it seems that the current Diagnostic Sheet needs to incorporate other indices more relevant to the structural resistibility. These indices may include "thickness of structural components," "weight of reinforcement bars" or "ratio of wall thickness to water depth." Factoring in these aspects could make the structural assessment more accurate and in turn improve the Diagnostic Sheet approach.
From this standpoint, we continue to collect more DEM test results on structures based on older SDCs, as well as the structural design information from latest-SCD-based structures. By analyzing these data through the current SEM and then comparing its results with the Revised SEM test results, we will appraise its applicability to assessing actual facilities.

CONCLUSIONS

The investigation into the damage by the Great East Japan Earthquake showed the facilities had suffered most from the ground failure due to liquefaction. On the other hand, the impact from other effects was almost insignificant.
The observations from this investigation and the case study made it possible for us to identify some weaknesses of the current Diagnostic Sheet. In response we proposed a Revised SEM procedure consisting of three steps factoring in the possibility of liquefaction, year of construction, and structural strength. Founded on the observations from the current SEM Diagnostic Sheet as well as the recent earthquakes, this approach will be simpler and easier-to-use than the current SEM, allowing for more practical assessment of "seismic preparedness" of the target structure.
One of the items for further consideration is to develop a more suitable assessment method of the structural resistibility. One promising revision is to add new indices which may allow for a more accurate structural analysis. After combining all these updates in the current framework of the Revised SEM, we will compile its methodology into a guidebook for practical application in small-scale water systems.
We hope this research and its fruit will contribute to promotion of earthquake proofing of drinking water infrastructure in Japan.

ACKNOWLEDGEMENT

We extend our sincere gratitude to everyone involved in this research project for their generous support.

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Damage to Water Supply Pipelines in the 2011 Great East Japan Earthquake

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ABSTRACT

This paper is focusing on damage to water supply pipelines during the 2011 great east Japan Earthquake. Severe damages were occurred not only to houses and buildings but also lifeline facilities including water supply facilities by this earthquake and tsunami. Water supply pipelines were damaged by strong ground motion and/or soil liquefaction. Outline of the damage is given first. Then the damage analysis is done by using GIS database of water supply pipelines and its damage. Lessons learned regarding the earthquake performance of water supply pipeline are summarized.

INTRODUCTION

The earthquake occurred on March 11, 2011 at 14:46 JST in the north-western Pacific Ocean at a relatively shallow depth of 32 km, with its epicenter approximately 72 km east of the Oshika Peninsula of Tohoku region, Japan. A JMA (Japan Meteorological Agency) seismic intensity of 7, that is, the maximum grade of JMA scale was recorded at K-NET Tukidate observation station in Kurihara City, Miyagi Prefecture. The earthquake generated a tsunami of unprecedented height and special extent along the coast of the main island of Japan. The earthquake and tsunami caused about 20,000 deaths and missing and injured about 6,000 people.

Water supply facilities were damaged severely and a suspension of water supply was occurred at about 2.57million houses in the wide area from Tohoku to Kanto regions just after the earthquake. This paper introduced an outline of the damage to water supply pipelines during this earthquake and the damage analysis done to the pipelines buried in liquefied areas and strong ground motion areas.

OUTLINE OF EARTHQUAKE AND DAMGE TO WATER SUPPLY FACILITIES

The earthquake was initially reported as 7.9 *Mw* by the United States Geological Survey (USGS) before it was quickly upgraded to 8.9 *Mw* then finally to 9.0 *Mw*. This earthquake is the fourth largest in the world since 1900 and largest in Japan recording started 130 years ago. Figure 1 shows a distribution of JMA seismic intensity. This figure indicates large seismic intensity over 6- was recorded at many

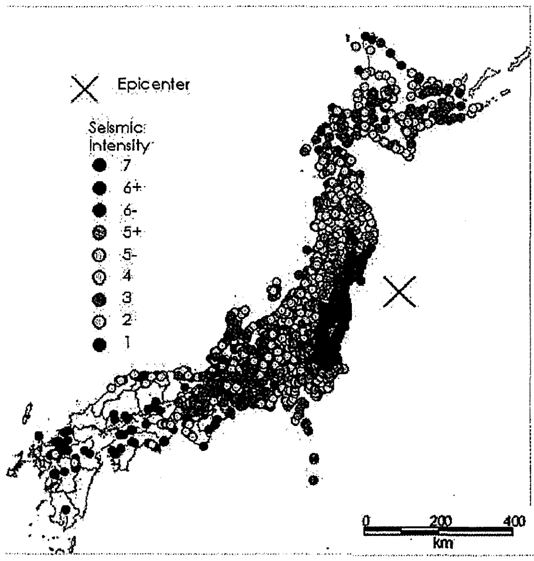

Figure 1. Distribution of JMA seismic Intensity (Japan Meteorological Agency).

Table 1. Large PGA observation stations (National Research Institute for Earth Science and Disaster Prevention).

	Site Name	Site Code	PGA (cm/s/s)	PGV (cm/s)
1	K-NET Tsukidate	MYG004	2,933	106
2	K-NET Siogama	MYG012	2,019	64
3	K-NET Hitachi	IBR003	1,845	74
4	K-NET Sendai	MYG013	1,808	83
5	K-NET Hokota	IBR013	1,762	71
6	K-NET Imaichi	TCG009	1,444	48
7	K-NET Shirakawa	FKS016	1,425	63
8	KiK-net Nishigou	FKSH10	1,335	41
9	K-NET Oomiya	IBR004	1,312	47
10	KiK-net Haga	TCGH16	1,305	82

PGA and PGV: Vectorial Summation of 3 Components
Bandpass filter to calculate velocity waveform : 0.1-15Hz

sites not only at Tohoku region but also at Kanto region. Table 1 lists observation sites where large peak ground acceleration (PGA) was recorded. PGA and PGV (peak ground velocity) listed here are vectorial summation of three components. The maximum PGA was 2,933 (cm/s/s) at K-NET Tsukidate observation station and the maximum PGV also recorded at Tsukidate. More than 1G (=980 cm/s/s) of PGA was observed at the nineteen observation stations of K-NET and KiK-NET observation networks.

Damage to water supply facilities was slight for the scale of the earthquake. Main facilities of water supply system are usually set in hill side far from sea. They did not suffer severe damage by the tsunami. Water resources and water pipe bridges on river near coastal area, however, suffered typical tsunami damages. Some points of ground water from well nearby coast were salinated for a long time and water pipe bridges were flowed out by the tsunami. The severest damage to water treatment facilities occurred by liquefaction. The typical cases happened in Chiba and Ibaragi Prefectures in Kanto region. Pipeline damage was also slight correspond to the past severe earthquake such as the 1995 Kobe Earthquake. This paper focuses on damage to pipeline buried in liquefaction area and strong ground motion area where the relatively severe damages to pipeline were occurred.

PIPELINE DAMAGE IN LIQUEFACTION AREA

Extensive liquefaction happened in Chiba and Ibaragi Prefectures in Kanto region. Especially liquefaction in filled land in Urayasu City of Chiba Prefecture caused large ground settlements and severe damage to buried pipelines. The to water supply pipeline damages in the filled land in Urayasu City is introduced here.

Figure 2 illustrates a water supply pipeline network and sites of damage to buried pipeline in a part of Urayasu City. The area in a yellow line in this figure is a filled land and severe liquefaction occurred in this area. The total piping length in this area was 200.93 km and the number of damage was 321, so the damage rate is 1.60 cases/km. The damage rate of pipeline buried in the reclaimed land of Kobe, Ashiya and Nishinomiya Cities in the 1995 Kobe Earthquake was 1.77 cases/km. So, degree of damage was seems to be similar. PGV of about 30 cm/s of Urayasu City was, however, much less than that at Kobe area in the 1995 Kobe Earthquake. So, degree of damage was seems to be similar. PGV of about 30 cm/s of Urayasu City was, however, much less than that at Kobe area in the 1995 Kobe Earthquake.

PIPELINE DAMAGE IN STRONG GROUND MOTION AREA

Maximum JMA seismic intensity of this earthquake was 7 in Kurihara City and the JMA seismic intensity in the surrounding cities and a town of Kurihara City was 6+. The damage to buried pipeline in Kurihara City and the surrounding cities and town, that is, Osaki City, Tome City and Wakuya Town is discussed here. Figure 3 illustrates distribution of JMA seismic intensity estimated by QuakeMap (National Institute of Advanced Industrial Science and Technology) and sites of damage to each pipe type of water supply pipeline in three cities and a town mentioned above. JMA seismic intensity estimated by QuakeMap is given in each 250 m cell. Table 2 lists a number of cells in each level of JMA seismic intensity of three cities and a town. 7.6% of the total area is 7 of JMA seismic intensity and 33.6% is 6+, 23.8% is 6- according to QuakeMap. Figure 4 illustrates an accumulated damage rate of each pipe type.

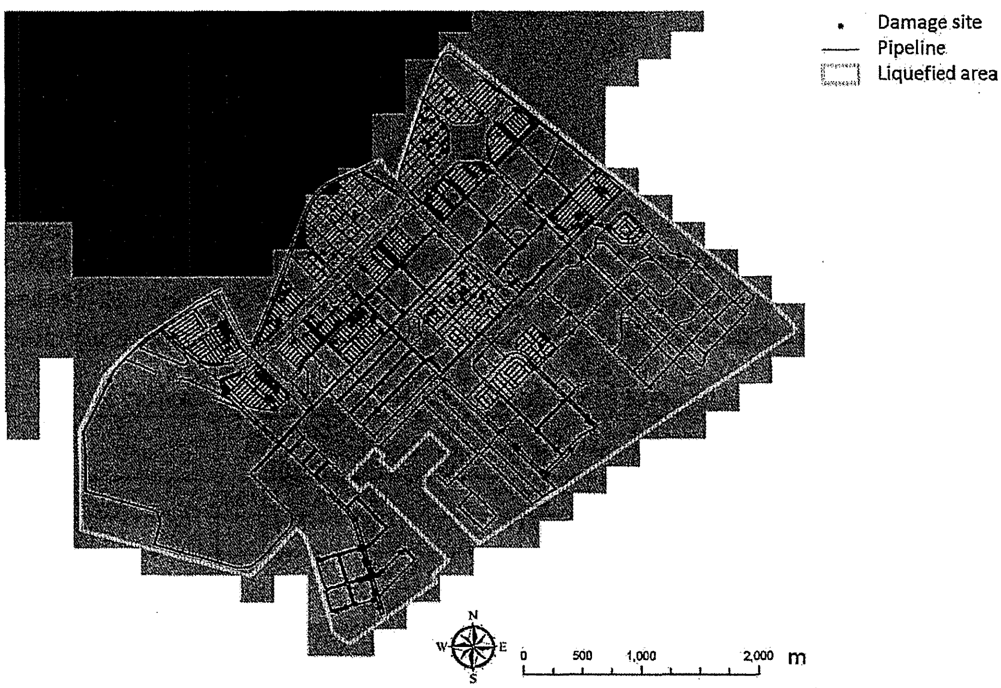

Figure 2. Water supply pipeline network and sites of damage to buried pipeline in a part of Urayasu City.

Figure 4 indicates that the damage to pipeline starts to occur at 5+ of JMA seismic intensity and the damage rapidly increases at 6+.

Micro topography classification map of J-SHIS surface ground (National Research Institute for Earth Science and Disaster Prevention) is used here. This data is also organized in 250 m cell. Micro topography classification is divided into two categories as same manner of "Handbook for help to determine earthquake-resistant compatibility of ductile iron pipe with K-type joints etc." shown in Table 3. Figure 5 illustrates distribution of ground categories, JMA seismic intensity and the sites of damage to each pipe type. Many damages occurred at the boundary between

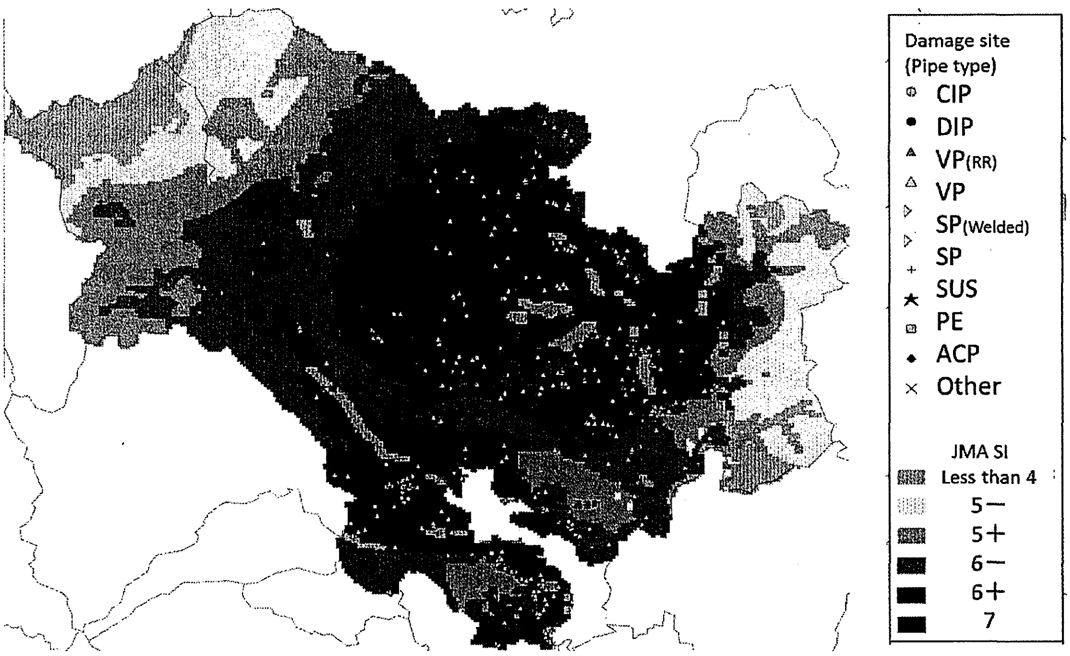

Figure 3. Distribution of JMA seismic intensity estimated by QuakeMap (National Institute of Advanced Industrial Science and Technology) and sites of damage to each pipe type of water supply pipeline.

Table 2. Number of cells in each level of JMA seismic intensity

JMA SI	Kurihara City	Osaki City	Tome City	Wakuya Town	Sum	Percentage
others	156	297	543	35	1,031	2.8%
less than 4	80	1,403	294		1,777	4.8%
5−	1,448	855	1,902	12	4,217	11.4%
5+	1,647	2,486	1,292	465	5,890	16.0%
6−	3,452	3,456	1,209	672	8,789	23.8%
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Sum	13,235	13,340	8,888	1,427	36,890	100.0%

bad of micro topography classification category and more than 5+ of JMA seismic intensity, and others. Figure 6 illustrates a comparison of damage rate of each pipe type in micro topography classification category. This figure suggests that the damage rates in category of bad are much greater than those in category of good. It is also clarified that the damage rate of pipelines buried in the strong ground motion area and micro topography classification category of bad was lower than that in liquefaction area such as in Urayasu City.

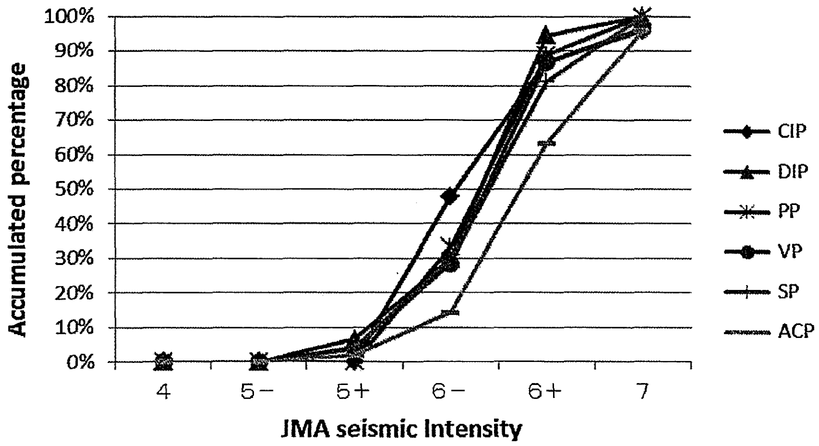

Figure 4. Accumulated damage rate of each pipe type.

Table 3. Micro topography classification and categories.

No.	Microtopography	Category
1	Mountains	
2	Piedmont areas	
3	Hill	
4	Volcanic areas	
5	Volcanic piedmont areas	Good
6	Volcanic hills	
7	Mesa	
8	Quality gravel plateau	
9	Rohm plateau	
10	Lowland valley	
11	Alluvial fan	
12	Natural levee	
13	Backswamp	
14	Old River Road	
15	Delta・ Coastal lowland	
16	Reef・ In gravel	
17	Dune	Bad
18	Reef・ Between the lowland dunes	
19	Reclaimed land	
20	Filled land	
21	Rocky・ reef	
22	Riverside	
23	River channel	
24	Lake	

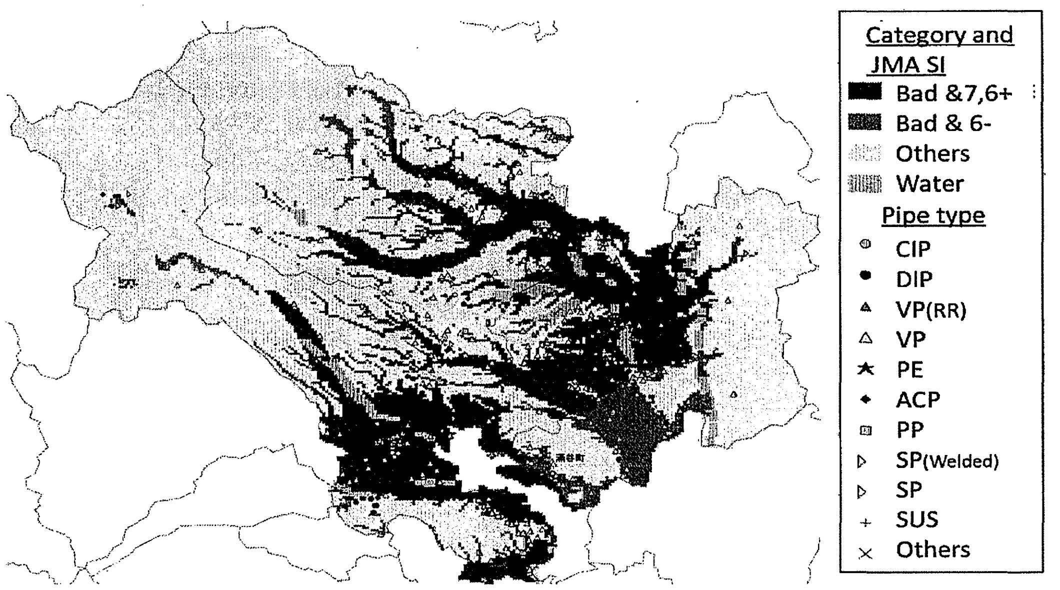

Figure 5. Distribution of ground categories, JMA seismic intensity and the sites of damage to each pipe type.

Table 4 lists a piping length in each level of JMA seismic intensity, categories of micro topography classification and pipe type. There was no damage to ductile iron pipe with earthquake resistant joint and polyethylene pipe with fusion bounding in this earthquake. According to Table 4, about 45km of earthquake resistant DIP and about 15km of PE with fusion bounding survived in the strong ground motion area, that is, JMA seismic intensity was more than 6- and micro topography classification category of bad.

CONCLUSION

An outline of the damage to water supply pipelines by the 2011 great east Japan Earthquake was given and the damage rate of pipeline was discussed. The following conclusions may be drawn based on the present study.

(1) The damage rate of water supply pipeline in filled land in Urayasu City was 1.60 cases/km. This value was similar to the damage rate of pipeline buried in the reclaimed land of Kobe, Ashiya and Nishinomiya Cities in the 1995 Kobe Earthquake.

(2) According to the data of Kurihara City, Osaki City, Tome City and Wakuya Town in Tohoku region, the damage to pipeline starts to occur at 5+ of JMA seismic intensity and the damage rapidly increases at 6+.

(3) The damage rates in micro topography classification category of bad were much greater than those in category of good.

(4) There was no damage to ductile iron pipe with earthquake résistant joint and polyethylene pipe with fusion bounding in this earthquake. About 45km of earthquake resistant DIP and about 15km of PE with fusion bounding survived in