國家地震工程研究中心

NATIONAL CENTER FOR RESEARCH ON
EARTHQUAKE ENGINEERING

Proceedings of the Sixth Taiwan-US-Japan Workshop on Water System Seismic Practices

October 14-15, 2009, Taipei, Taiwan

Edited by

Tsung-Shen Liao, Keh-Chyuan Tsai and Liang-Chun Chen

Report Number：NCREE-09-019

October 2009

NCREE-09-019

Proceedings of the Sixth Taiwan-US-Japan Workshop on Water System Seismic Practices

October 14-15, 2009, Taipei, Taiwan

Edited by

Tsung-Shen Liao
President of Chinese Taiwan Water Works Association

Keh-Chyuan Tsai
Director of National Center for Research on Earthquake Engineering

Liang-Chun Chen
Director of National Science and Technology Center for Disaster Reduction

Workshop Co-Sponsored by

Chinese Taiwan Water Works Association
Water Research Foundation, USA
Japan Water Works Association
National Center for Research on Earthquake Engineering, Taiwan
National Science and Technology Center for Disaster Reduction, Taiwan
Water Resources Agency, Ministry of Economic Affairs, Taiwan
Taiwan Water Corporation
Taipei Water Department
Sinotech Engineering Consultants, Ltd., Taiwan
CECI Engineering Consultants, Inc., Taiwan

October 2009

Preface

The goal of the Sixth Taiwan-US-Japan Workshop on Water System Seismic Practices aims at providing an avenue for practitioners and researchers to share knowledge and experiences in the field of seismic safety, preparedness and serviceability of water works. This year is the tenth anniversary of the 1999, Chi-Chi Taiwan earthquake. Therefore, it is very meaningful for the Chinese Taiwan Water Works Association to have the opportunity to host the Workshop in Taipei. The proceedings contain all papers to be presented during the Workshop. They can be categorized into four topics: (1) seismic mitigation measures, (2) seismic analysis, design and retrofit, (3) seismic risk analysis, assessment and management and (4) emergency response and recovery. Finally, I would like to thank all speakers and attendees for their participations in the Workshop, and I do believe that through the sharing and friendship established in this activity, we can promote further exchange and collaboration among Taiwan, US and Japan in the field of water system seismic practices.

Tsung-Shen Liao, Ph.D.
President, Chinese Taiwan Water Works Association
2009.10.14, Taipei

Contents

Water Supply Facilities Damages by the 1999 Chi-Chi Earthquake and the Aftershock Restoration Works

Tsung-Shen Liao[1]

I. INTRODUCTION

Ten years ago, the most devastating earthquake in the 20th Century for Taiwan, the Chi-Chi earthquake, caused tremendous damages to the country, claiming over 2,500 lives and 10.7 billion USD direct property loss. Damage below the ground surface caused by the Chi-Chi earthquake that Taiwan experienced at the midnight of September 21, 1999 was just as devastating as the structural damage above the surface. Among the destructions induced by the earthquake, the water supply systems endured a lot of breakages and failures. Both the water treatment facilities and underground pipelines suffered widespread damages. From the investigations conducted by the National Science Council of ROC, many water treatment facilities in the affected areas were out of order after the earthquake. The major causes for break and ruptures of water pipelines were vibration/ground shaking (48%), vertical ground movement (16%), and ground collapses (11%) with other minor factors including ground cracking and opening, horizontal ground movement, and liquefaction.

Systems of buried pipelines, which brought potable water to the cities and counties near the earthquake epicenter, were totally down due to serious damage. The induced damage resulted in the failure of pipeline systems to provide water for drinking and fire protection. Due to the close relationship to health and daily life, the subsurface damage of the water pipelines posed a serious threat to an untold number of residents living in central Taiwan. After the Chi-Chi earthquake, shock damaged water pipelines were pervasive around the area close to the epicenter, including Taichung, Nantou and Changhua. The field investigation involves observations on pipelines of various materials and types of joints in the water supply system near the earthquake epicenter. Comprehensive comparisons are made between the observed damage and that experienced in two previous major earthquakes, the 1976 Tangshan earthquake *(M=7.8)* and the 1995 Kobe earthquake *(M=7.I)*. The field investigation consequently led to an experimental program, in which some laboratory tests were conducted to have a preliminary understanding of the seismic resistance of pipe joints locally used. Based on the test results, some recommendations on measures against earthquake strikes are accordingly made.

II. THE WATER SUPPLY SYSTEM OF TAIWAN

To serve the ever-growing population in Taiwan (except Taipei), the pipeline system of Taiwan Water Corporation (TWC) daily supplies about 9 million cubic meters of fresh water for more than 85% of the population in Taiwan. The raw water of Taiwan mostly comes from seasonal rainfalls over the central ridge mountain area and is recovered by

[1] President, Chinese Taiwan Water Works Association

pumping it from many dams, reservoirs, rivers and deep wells. After treatment and storage, the fresh water is fed to the widely spread cities through long-distance aqueducts and pipelines. Currently, the TWC operates and maintains more than 55,000 kilometers of water lines: main aqueducts, large-diameter pipelines and various distribution lines with diameters greater than 50 mm. In the Chi-Chi earthquake, the induced pipeline damage influenced nearly half of the TWC water supply system.

III. DAMAGE TO THE WATER PIPELINES

As experienced in previous earthquakes, for instance the 1985 Mexico earthquake, the area of the subsurface damage to water pipelines is much larger in size than the area of above-surface structural damage. This is also one of the most striking aspects of the Chi-Chi earthquake damage to the water supply system. The site investigations approximate a total length of damaged water pipelines of around 16,000 km, including 12,000 km of primary water supply lines, in the damage areas mentioned above.

IV. EFFECTS OF PIPELINE MATERIALS AND JOINTS

A variety of materials, including SP, CIP, DIP, PE and PVC, are used for the pipelines of the TWC water supply system, and consequently different modes of damage were observed in the Chi-Chi earthquake. As the field observation indicates, the steel pipes (SP) of high tensile strength and the polyethylene (PE) pipes of good ductility were both received only small damage in the Chi-Chi earthquake. On the other hand, since polyvinyl chloride (PVC) pipes are brittle and used for smaller diameter lines, there were many PVC pipelines broken during the intensive ground shake. It is found that the joint of the PVC pipe seems very vulnerable in the earthquake. For other types of pipe, such as the DIP pipes, the damage to segmented pipes most often occurs at the joint too. The DIP joint is of the rubber gasketed type with "slip" characteristics, and there are some damage cases for this particular type joint. In fact, the damage investigations of the water supply pipeline in this and others major earthquakes, previous, have indicated that the amount of joint damage statistically accounts for at least a half of the total resulting damage. Measures for improving the seismic resistance of pipeline joints are surely valuable for future research.

V. SUMMARY

Immediately after the earthquake, herculean efforts from TWC mostly invested to the repairing works. Continuing for weeks, all the efforts were organized on a round-the-clock basis. Before damage recovery, TWC devised the emergency water supply using portable reservoirs served by tank trucks bringing water to provide enough clean water to meet the needs of the stricken areas. In the mean time, damage assessment indicates that serious damage to water supply pipelines in the Chi-Chi earthquake can be attributed to many factors involving at least: (1) the irresistible faulting movement along the fault zones. (2) the intensive ground shaking near the epicenter. (3) the site amplification of ground shaking induced by varying local geology. (4) the liquefaction due to poor ground condition.

The investigation also concludes that damage modes of the buried water pipelines can be categorized in accordance with different materials of the pipelines. The damage to the pipeline was effected by the soil-pipeline interaction, which depended mainly upon the pipeline diameter. Also, the field investigation results reveal that the pipe fitting connections are apt to suffer damage, and the joint of the pipe is the key point of earthquake resistance.

Integration of Seismic Modeling Software with Emergency Response Software

Xavier Irias[1] and David Wald[2]

ABSTRACT

This paper discusses a method for integrating estimates of seismic intensity and resulting facility damage with generalized emergency response tools and procedures. The integrated method combines readily available seismic modeling software (e.g., *ShakeCast*[1]) with in-house developed Emergency Information Management (EIM) software.

While the two types of software tools, seismic modeling and emergency response, clearly provide value when used separately, they provide more value if integrated. Integration simplifies and streamlines damage assessment and response, shaving minutes or hours off response times; it also reduces system costs.

East Bay Municipal Utility District (EBMUD), a utility in the San Francisco Bay Area, provides water service to 1,300,000 customers and wastewater service to about 600,000 customers. With over 4,000 miles of water pipe and hundreds of major facilities in an area of high seismicity, EBMUD has invested in seismic modeling, seismic readiness, and emergency response. EBMUD is advancing these functions by integrating *ShakeCast* with its in-house EIM software.

ShakeCast, produced by the United States Geological Survey (USGS) and freely available, provides earthquake intensity and facility damage estimates based on scenario or actual earthquakes. While *ShakeCast* can be used anywhere in the world, other tools, for example, the Japanese National Research Institute for Earth Science and Disaster software *J-SHIS*[2] could be used if modeling scenarios in Japan.

Marconi, EBMUD's in-house EIM software, is similar to commercial EIM software in that it facilitates a general-hazard response. It differs in that it is fully customizable to integrate with the *ShakeCast* software and other similar tools, providing the following benefits over using the software components standalone:

1. Promotes a multi-hazard perspective so all risks are considered appropriately, and selected mitigations better promote overall system resiliency.
2. Provides valuable hands-on experience with the EIM *before* a disaster strikes.
3. Saves money through reduction of labor and consolidation of data.
4. Improves the speed and effectiveness of emergency response by leveraging the analytical capabilities of the EIM software with seismic modeling results.

This paper discusses the design of the integrated system and provides information on how other utilities could integrate seismic modeling efforts with emergency response tools.

[1] Director of Engineering and Construction, East Bay Municipal Utility District.
[2] Adjunct Associate Professor of Geophysics Colorado School of Mines;
Supervisory Research Geophysicist, National Earthquake Information Center, United States Geological Survey, Golden, Colorado.

INTRODUCTION

About EBMUD

EBMUD is a utility in the San Francisco Bay Area that provides water service to 1,300,000 customers and wastewater service to about 600,000 customers. EBMUD owns and operates over 4,000 miles of water pipe and hundreds of major facilities in an area of high seismicity and has, for many years, worked on seismic modeling, seismic readiness, and emergency response. Figures 1 and 2, respectively, show the location of EBMUD's service area and the faults within the service area.

Figure 1. Location of EBMUD's Service Area

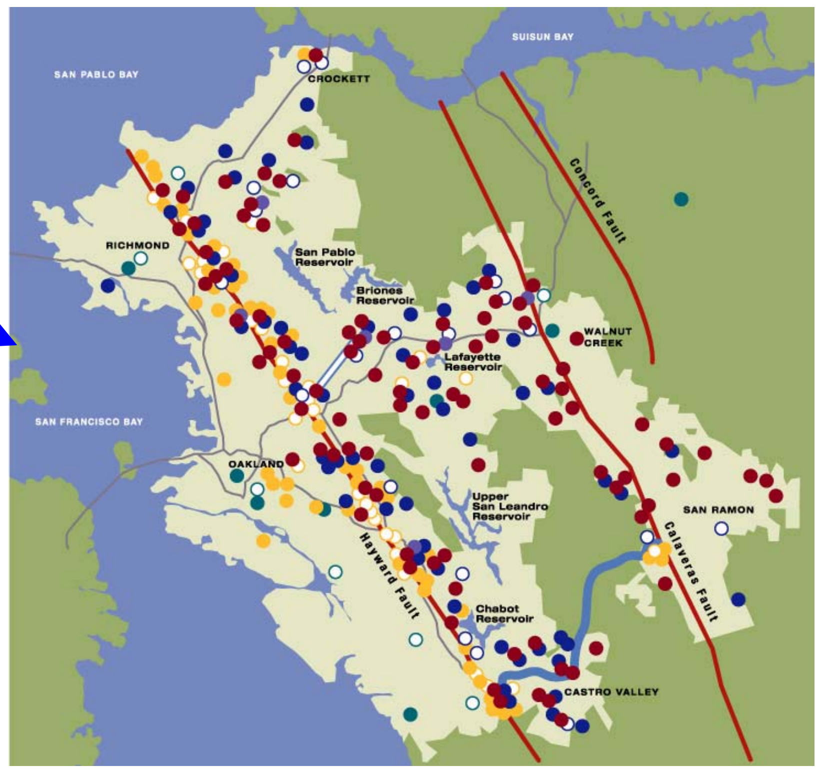


Figure 2. EBMUD Service Area Faults (Markers Represent Facilities)

Seismic Setting

The highly active Hayward Fault dominates EBMUD's seismic risk profile. This fault, capable of earthquakes of magnitude 7.0, has produced major earthquakes on average every 140 years, with the last major earthquake occurring in 1868 – in other words, if past historic patterns continue, the next major earthquake is due any time. The economic losses from a similar earthquake occurring today have been estimated at up to $200 billion United States Dollars[3]. As shown in Figure 2, dozens of EBMUD's critical facilities are located within a few hundred meters of the Hayward Fault.

Beyond the Hayward Fault, several other faults threaten EBMUD's system, ranging from the San Andreas Fault west of the service area to the Calaveras Fault in the eastern portion of the service area.

Prior Seismic Modeling and Upgrades by EBMUD

Historically, EBMUD has performed seismic modeling to identify and address system deficiencies. That modeling has been very valuable to pinpoint where capital dollars should be invested to maximize benefits to our customers in terms of safety and reliability. The 10-year, $200 million Seismic Improvement Program (SIP), which was completed in 2006, was based on modeling the system's response to scenario earthquakes. In the course of that program, EBMUD upgraded dozens of facilities and installed fault crossing connections at key locations along the Hayward Fault.

The SIP was tremendously successful; by one estimate, it will reduce economic losses in the region by about $2 billion in the likely event that a major earthquake occurs in the next 30 years. However, it remains likely that a major earthquake will inflict severe damage on EBMUD's systems and on other lifeline dependencies such as electrical power. While strengthening the system to preclude damage is infeasible, measures are being taken to reduce the likelihood of component failures as well as the consequence of those failures on system performance and hence impact to our customers. EBMUD's strategy for doing so is discussed next.

Strategy

The overarching strategy elements are:

1. Consider seismic performance when making capital improvements. This consideration needs to be given when selecting and prioritizing projects, as well as during the scoping and design of individual projects.
2. Be prepared to reduce the impact of component failures after an earthquake. To carry out this part of the strategy, we need to be ready to quickly assess system damage, then to prioritize and implement response and recovery actions.

Seismic modeling is clearly part of the first strategy element. For example, earthquake scenario modeling can identify vulnerable elements in the system and elevate the priority of remedial projects. And, it can help determine the scope of projects after they are selected for implementation.

Perhaps less established is the extent to which seismic modeling can help with the second strategy element, i.e., with quick post-disaster response. The principal topic of this paper is to explain why and how one can integrate seismic modeling into emergency planning and response.

BENEFITS OF SEISMIC MODELING FOR EMERGENCY PLANNING AND RESPONSE

Seismic scenario modeling can provide guidance to one's capital investments regardless of whether that modeling is tied explicitly to emergency planning procedures. *What then are the additive benefits of integrating the seismic modeling effort with emergency planning and response tools?*

Benefits to the Planning Process

The benefits of integration to the scenario planning process include:

1. Promotion of a multi-hazard perspective.
2. Improved user familiarity and competency with tools.
3. Reduced cost for training and data maintenance.

Each is discussed briefly below.

Multi-hazard Perspective

Incorporating seismic modeling into the emergency planning process promotes a broader perspective than that afforded by seismic damage modeling in isolation. Incorporated into a multi-hazard emergency plan, earthquakes are considered not as the sole hazard, but as one of many possible hazards. This allows all of the various hazards to be viewed in perspective so that, for example, wildfires do not receive all of the planning effort at the exclusion of earthquakes, or vice versa. A multi-hazard perspective also promotes concepts of resiliency and robustness, rather than narrowly focused scenario-specific mitigations. As an example, a multi-hazard planning process may identify certain mitigations, such as improved communication tools, that mitigate the effects of a wide range of hazards and promote resiliency or flexibility for as-yet unidentified hazards. By contrast, an approach that focused on a particular hazard type such as an earthquake would not tend to value such cross-cutting mitigations appropriately, and could thus overemphasize mitigations that were not resilient in the face of a wider range of hazards. For example, structural strengthening mitigates for earthquakes, but does nothing for widespread power loss or a cyber attack.

The benefits of an all-hazard perspective may seem obvious, but nonetheless some theoretical work has been devoted to it. An all-hazard perspective is an example of Robust Decision Making, or RDM[4], as promoted by the RAND Corporation. RDM involves studying the performance of a system when subjected to a wide range of scenarios, with the goal that investment decisions for system improvement or mitigations are "robust" no matter what scenarios ultimately occur. Multi-hazard scenario development as a building block for RDM thus has the potential to improve system robustness.

Improved User Competency

Another benefit of integrating seismic modeling tools with emergency planning and response tools is that users gain familiarity and competency with emergency response software tools. This is invaluable since the experience gained using the emergency response tools will lead to more accurate and efficient use following a disaster.

Reduced Cost

A third benefit during the planning stage is reduced cost through consolidation. By integrating the software tools, users need only learn a single software package, and do not need to train and practice with multiple software packages, and IT support personnel need to support a smaller number of systems.

Data maintenance costs are also reduced. In particular, there is no need to manually maintain facility databases in both *Marconi* and *ShakeCast*.

The integrated approach thus yields combined savings in hardware, software licensing, system administration, training, and data maintenance, which multiply over the life of the system, resulting in potentially very significant savings.

Benefits Following a Disaster

While the pre-disaster benefits are substantial, as explained above, the greatest benefits of integration will accrue following a disaster. At that time, every minute matters as responders struggle to gain situational awareness. An integrated emergency tool that can overlay seismic information along with all of the other salient information is invaluable to providing a coherent and rapid response following an emergency.

It is true that *ShakeCast* or a similar tool could be used even absent integration to gain "situational awareness," or an overall assessment of a system's status. For example, *ShakeCast* can produce listings of facilities with potential damage rankings, and those listings can help prioritize inspections. The gain from integration of *ShakeCast* with an EIM is the ability to immediately combine and overlay data of all types coming in from the field, in both tabular and graphic forms. Table I below is a sample to illustrate this concept. Note that the decisions regarding inspection priority are influenced by modeled exposure, but the results of the inspection ultimately trump the modeling information.

TABLE I. SAMPLE OF INCOMING FIELD DATA

Facility	Type	Status	Earthquake exposure	Notes
Acme WTP	Water treatment plant	Inspection pending	PGA=.6g Moderate damage potential	Inspector Curly on his way; top inspection priority given its exposure level
Blackstone Pumping Plant	Pumping Plant	Inspection scheduled for tomorrow	PGA=0.3g	Non-critical, minimal exposure, so inspection can wait
Dunderhill Dam	Dam	Inspection completed, verified no damage	PGA=0.8g Slight damage potential	Inspection performed asap because of criticality and exposure; cleared

SYSTEM ARCHITECTURE

Seismic Modeling Component

The seismic modeling component can be viewed as a black box that produces grids of earthquake intensity for actual earthquakes and/or scenario earthquakes. Typically, the grids are produced as text files, specifically either XML or CSV format. An excerpt of a *ShakeCast* grid file, xml-formatted, is shown below:

```
<shakemap_grid xsi:schemaLocation="http://earthquake.usgs.gov
http://earthquake.usgs.gov/eqcenter/shakemap/xml/schemas/shakemap.xsd" event_id="2009ixae"
shakemap_id="2009ixae" shakemap_version="2" code_version="3.2.1 GSM"
process_timestamp="2009-07-10T01:15:20Z" shakemap_originator="us" map_status="RELEASED"
shakemap_event_type="ACTUAL">
<event magnitude="5.3" depth="381.6" lat="47.909200" lon="148.248300"
event_timestamp="2009-07-10T00:49:10GMT" event_description="NORTHWEST OF THE KURIL ISLANDS"/>
<grid_specification lon_min="146.264967" lat_min="46.585867" lon_max="150.264967"
lat_max="49.252533" nominal_lon_spacing="0.083333" nominal_lat_spacing="0.083333" nlon="49"
nlat="33" nominal_pga_std="0.920000"/>
<grid_field index="1" name="LON" units="dd"/>
<grid_field index="2" name="LAT" units="dd"/>
<grid_field index="3" name="PGA" units="pctg"/>
<grid_field index="4" name="PGV" units="cms"/>
<grid_field index="5" name="MMI" units="mmi"/>
<grid_field index="6" name="PSA03" units="pctg"/>
<grid_field index="7" name="PSA10" units="pctg"/>
<grid_field index="8" name="PSA30" units="pctg"/>
<grid_field index="9" name="SDPGA" units="none"/>
<grid_field index="10" name="SVEL" units="ms"/>
-
<grid_data>

146.264967 49.252533 0.1449 0.4267 1.3300 0.2329 0.4507 0.0197 1.0000 360.0000
146.348300 49.252533 0.1465 0.4308 1.3400 0.2354 0.4551 0.0199 1.0000 360.0000
146.431633 49.252533 0.1481 0.4349 1.3500 0.2379 0.4594 0.0201 1.0000 360.0000
146.514967 49.252533 0.1497 0.4388 1.3600 0.2402 0.4635 0.0203 1.0000 360.0000
```


A schematic grid is shown graphically as Figure 3 below. The actual grids from *ShakeCast* are finer, typically with resolutions of less than 0.10° (the example above is 0.0833°), and have hundreds of rows in each direction.

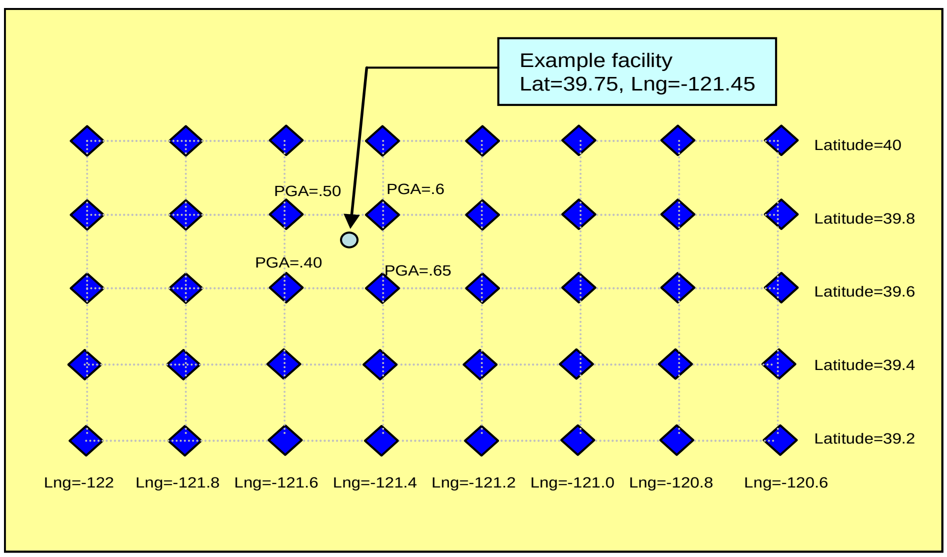


Figure 3. Schematic Shakecast Grid

While *ShakeCast* is capable of modeling facility damage on its own, the emergency software, *Marconi*, does not rely on that feature. Instead, *Marconi* computes facility damage from the *ShakeCast* grid of earthquake metrics as shown above. This is a fairly simple procedure consisting of bilinearly interpolating [3] the various earthquake metrics at each facility location. The appropriate metrics are then compared to each facility's fragility data to determine the potential damage state of each facility. Figure 4 below shows a sample fragility curve, in this case, for a freeway bridge owned by Caltrans, an agency responsible for thousands of major structures in the seismically active State of California.

3 Interpolation differs from *ShakeCast's* standard method of taking the maximum value among the four surrounding grid-points. Clearly, interpolating reduces the mean error, while grabbing the maximum value could be considered more conservative.

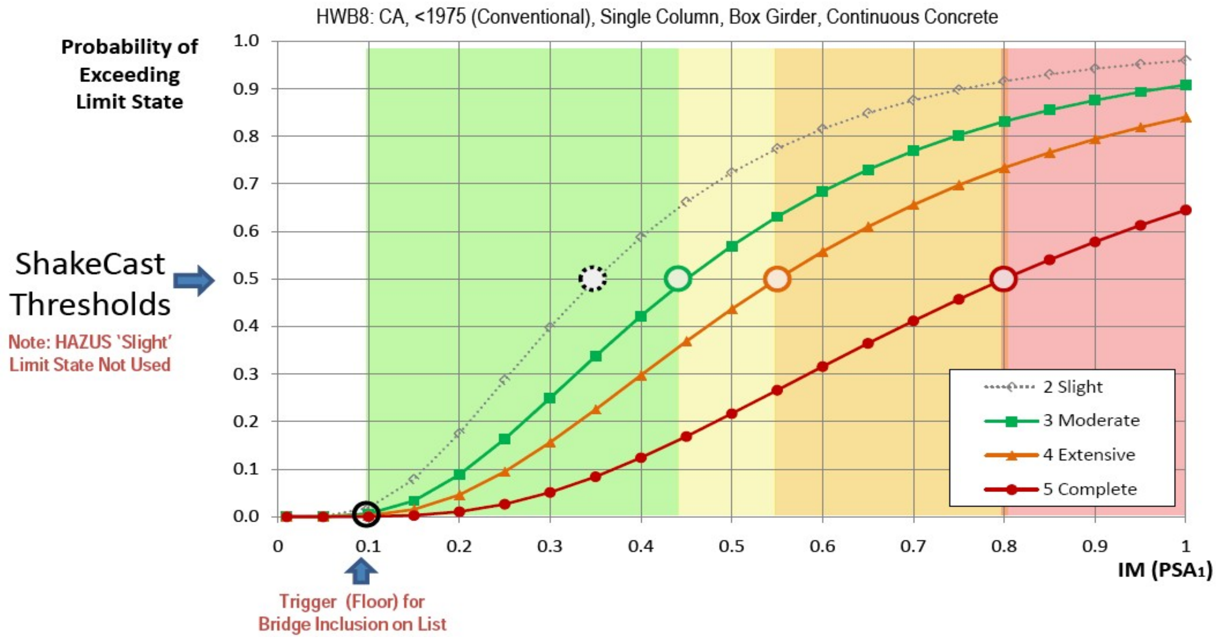


Figure 4. Facility Fragility Example (Courtesy of Caltrans)

The science of aboveground structural fragilities appears to be further developed than that for buried structures, including pipelines. The appropriate fragility formulations to use for buried assets remain a subject of considerable ongoing interest, given their tremendous value and their vulnerability to damage[5]. An advantage of bringing the damage calculation into *Marconi* rather than leaving it to *ShakeCast* is to allow a range of facility damage algorithms to be readily explored. It also obviates the need to keep the *ShakeCast* facility list in sync with that of *Marconi's*.

The user need interact only with *Marconi*. Following an earthquake, typically the *Shake map*, as a grid file, will have been automatically pulled from USGS servers by either *ShakeCast* or *ShakeCast Lite*. The *Marconi* administrator simply invokes the "upload quake metrics" menu option in *Marconi* and uploads the grid file to *Marconi*. Within seconds, *Marconi* has processed the grid and assigned damage metrics to each geo-coded facility for which fragility data are configured.

Emergency Planning and Response Component

Once the earthquake data are uploaded to *Marconi*, which takes a matter of seconds, the modeled facility damage is available along with data from the field, such as damage reports. *Marconi* provides listings of facility status that incorporate field reports along with the modeled level of seismic exposure. This allows emergency responders and decision makers to make judgments based on the best combination of data. Very soon after an earthquake, they will tend to rely more on modeling data than field data; as time goes on, the field data will tend to supersede the modeling data in importance. Integrating the seismic modeling data into the emergency response system allows a smooth transition from reliance on modeling results to reliance on actual damage reports.

The Marconi system is, at its heart, a relational database. Relational databases are highly mature technologies that are scalable in many ways, including speed, capacity, concurrency and availability, and are relatively easy to integrate with other systems.

Marconi's database includes well over a hundred tables, but the core tables number only a few, corresponding to key *Marconi* entities, as shown in the sidebar "*Marconi* Key Entities."

Marconi Key Entities

Crisis: Top-level event that triggers an emergency. A crisis could be an earthquake, a hurricane, pandemic, or any other disastrous event. A crisis may be linked to a predefined scenario.

Scenario: A hypothetical situation, such as an earthquake at a particular location of a particular magnitude. *Marconi* allows detailed scenario plans to be developed.

Incident: Occurrence caused by a crisis. Example incidents are main breaks, power outages, structural failures, and chemical spills. Incidents are geo-coded, prioritized, and classified several different ways.

Facility: A fixed asset, whether part of an agency's infrastructure, such as a water treatment plant, dam, pumping plant or pipeline; an important customer facility, such as a school or factory; or a key dependency, such as a power substation. Key attributes include the operational status and modeled damage.

User: Usually an employee, but could be any authorized person. Keeping user status up-to-date is critical since many users will likely be displaced or injured by a major crisis or may be needed at home.

Resource: A resource is a movable asset, such as equipment, materials or supplies, whether owned by the primary agency or provided by mutual aid.

As indicated in the sidebar, one of *Marconi's* focus areas is on facilities, corresponding to an organization's need following a disaster to quickly assess facilities. In the context of earthquake hazards, key attributes of a facility include the facility's location and its seismic fragility. That information, together with a *ShakeCast* grid file, is used in the first minutes after an earthquake to estimate damage potential at a facility. Facility damage can also be estimated prior to a disaster, for scenario development purposes.

User-related information is also a focus area since employee status may be unknown and communications impaired, so standard work processes will be insufficient to assign and track work. For example, after Hurricane Katrina, up to 75 percent of needed employees did not report to work for weeks following the event[6]. *Marconi* addresses this need by making it easy for employees to check-in, receive and update work assignments, and update their contact information. This allows emergency managers to have an accurate picture at all times of who is available for work and who is doing what.

EXAMPLE SYSTEM USE CASE

To make the system description more concrete, a sample use case is given that covers a timeline beginning with a magnitude 7.0 earthquake in the heart of EBMUD's service area.

Assumptions

1. *ShakeCast* or *ShakeCast Lite* have been installed on EBMUD's systems and pre-configured to send a basic "grid" file to multiple destinations.
2. A *Marconi* server, located outside the damage zone, has been pre-populated with EBMUD's facilities.

Event

1. An earthquake of magnitude 7.0 occurs on the Hayward Fault at midnight on a weekday.
2. The earthquake causes massive damage to EBMUD facilities, to power facilities, and to some internet routers; however, hardened sites, including EBMUD's service centers, retain internet connectivity and backup power.

Timeline

Time T hours:minutes after event	Action
0:00	Earthquake occurs.
0:30	Employees awakened by the earthquake check on their own families, and try to contact the office, but find phone service is disrupted.
0:30 to 1:00	Employees begin to act out pre-designated roles. Some can simply return to bed. Others begin inspection duties. Still others drive to their designated location to check-in.
1:00	First employees begin check-in at facilities. This consists of logging into *Marconi* via the hardened internet facilities at service centers. In most cases, the employee returns home after checking in.
1:05	Designated engineers use *Marconi* to process *ShakeCast* information provided by USGS high-availability server. They inform the Emergency Operations Director of the earthquake's location and magnitude. Those engineers also produce a sorted list of high-priority inspections based on the modeled damage. Attempts are made to contact inspectors (via *Marconi*'s notification service) to adjust their inspection priorities accordingly.
1:30	First inspection reports become available. Those reports are logged into *Marconi* – either by using one of EBMUD's hardened service centers to access the internet or, if possible, with mobile devices such as smartphones acting as *Marconi* clients.
1:45	*Marconi* produces maps overlaying facility condition per field inspection reports, along with modeled damage potential and reported incidents. Critical facilities still awaiting inspection, which are located in areas of high damage potential and/or in areas of high incident reports, are further escalated in priority and personnel dispatched accordingly.
2:00	Field reports of damage are generally correlating with model results. Accordingly, some operational and maintenance field crews are given instructions based not on observed damage, but on likely damage. For example: 1. Certain isolation valves are closed in order to retain storage based on a presumption that downstream facilities may have major leaks. 2. Treatment plants serving areas of high modeled damage are told to increase disinfection as a precaution. [Note: these actions are hypothetical and not yet part of any Standard Operating Procedure.]
3:00	The critical facilities targeted for high-priority inspection based on seismic modeling information have all been inspected and meaningful estimates of damage to the backbone systems can begin. *Marconi* produces lists of facilities showing operational status. Incidents such as facility damage, main breaks, etc., can now be prioritized based on the goal of preserving maximum system-wide capability. Inspections of the lower-priority, less likely damaged facilities are scheduled for first daylight (6:00 a.m.), and inspections tasked to individuals via *Marconi*.

Time T hours:minutes after event	Action
6:00	Employees continue to check in via *Marconi*. Dozens of employees have "assigned work" awaiting them upon login, primarily inspection of any facilities not already inspected. They use *Marconi* to immediately acknowledge the assignments prior to starting their assignments.
6:00 to 12:00	*Marconi* provides an evolving realtime picture of system status as all inspections are completed and logged. By the end of this period, the seismic model damage is no longer necessary, as it has been superseded by visual inspection in most cases.

APPLICATIONS BY OTHERS

Not only are the concepts discussed herein widely applicable, so are the tools. All of the software discussed is being made available to other agencies for their use and modification.

CONCLUSION

While seismic modeling tools and emergency response systems can each provide value when used separately, they are even more powerful when combined. Their integration provides a host of benefits, including better scenario planning, simpler use, and reduced costs. Most importantly, the integration of tools promotes quicker and more effective response to a major disaster by shaving precious moments off response time and improving accuracy of information. That improved response is essential to fulfilling our mission of providing reliable water supply to our customers.

REFERENCES

[1] http://earthquake.usgs.gov/resources/software/shakecast.

[2] http://www.j-shis.bosai.go.jp/doc/html/probability_strong_en.htm.

[3] http://www.strategicrisk.co.uk/story.asp?sectioncode=23&storycode=375172.

[4] Lempert, Robert J.; Groves, David G.; Popper, Steven W.; Bankes, Steven C. *A General, Analytic Method for Generating Robust Strategies and Narrative Scenario.* Available for download at http://www.rand.org/pubs/external_publications/EP20060412/.

[5] *American Lifelines Alliance Seismic Guidelines for Water Pipelines.* Available for download at http://www.americanlifelinesalliance.org/pdf/SeismicGuidelines_WaterPipelines_P1.pdf.

[6] Adachi, Takao. *Impact of Cascading Failures on Performance Assessment of Civil Infrastructure*, May 2007 thesis dissertation, Georgia Institute of Technology.

Recent Topics Related to Seismic Safety of Dams

Tatsuo Ohmachi[1]

ABSTRACT

Guidelines for Seismic Safety Evaluation of Dams (draft) issued in Japan is briefly introduced first. Damage to several dams of various types caused by recent earthquakes is overviewed next with some lessons learned. The earthquakes are the 1999 Chi-chi earthquake in Taiwan, the 2004 Niigata-Chuetsu and the 2008 Iwate-Miyagi inland earthquakes in Japan, and the 2008 Wenchuan earthquake in China.

INTRODUCTION

In Japan, "Guidelines for Seismic Safety Evaluation of Dams against Large Earthquakes (Draft)" was issued in March 2005 by the Ministry of Land, Infrastructure and Transport (MLIT) [1], which is called the Guidelines hereafter. Main target of the Guidelines is the introduction of the Level 2 earthquake motion to seismic safety evaluation of dams.

In principle, the Guideline is based on the "Proposals on Earthquake Resistance for Civil Engineering Structures", issued by the Japan Society of Civil Engineers (JSCE) following the 1995 Hyogoken-nambu earthquake which is often called the 1995 Kobe earthquake for short. One of the requirements of the JSCE proposals was to consider two levels of input motions for seismic design of structures. Level 1 (L1) covers motions of moderately high intensity while Level 2 (L2) addresses motions of extremely high intensity of the nature of the strong motion experienced in Kobe city during the 1995 earthquake.

The issue of the Guidelines was preceded by the 1999 Chi-chi earthquake, Taiwan, and the 2004 Niigata-ken Chuetsu earthquake, Japan, that caused serious damage to concrete and fill dams. These earthquakes together with the Kobe earthquake demonstrated urgent necessity of introduction of the L2 motions. In 2008, two damaging earthquakes occurred in Japan and in China. The one was the Iwate-Miyagi inland earthquake, and the other the Wenchuan earthquake.

Based on these facts, outline of the guidelines are briefly introduced first, followed by the earthquake damage to dams. Although the author was given much information to be described in this paper, he is solely responsible for any errors and misunderstanding.

OUTLINE OF THE GUIDELINES

The L2 motion is an input motion for seismic safety evaluation, having the highest intensity reasonably expected to occur at the dam site under consideration. The L2 motion is a so-called source specific, site-specific and structure specific motion. Uncertainty is inherent in the evaluation of L2 motion. The uncertainty lies not only in the rupture process of future seismic faulting, but also in the location of seismic faults.

[1] Tasuo Ohmachi, Professor, Department of Built Engineering, Tokyo Institute of Technology, 4259-G3-2, Nagatsuta, Midori-ku, Yokohama 226-8502, Japan

In fact, present maps of active faults do not contain blind faults. However, inland earthquakes caused by blind faults would affect dams. According to recent studies, inland earthquakes of magnitude M_J smaller than or equal to 6.5 are not likely to reveal their fault traces on the ground surface, even if the surface is not covered with sedimentary layers. On this basis, an inland earthquake caused by blind faults is thought to be a scenario earthquake, which gives a lower bound for the L2 motion.

For the L2 motion, structures are generally allowed to undergo plastic deformation as long as serious damage is prevented. The serious damage should include not only direct or structural damage to a dam body and appurtenant structures but also indirect one in consequence of collapse like loss of life in the downstream. Hence, regardless of dam type, requirement for dam safety against the L2 motion is stated as:

A dam is required (1) to maintain its capability of water storage during as well as after the L2 earthquake motion, and (2) to remain within repairable damage even if it suffers earthquake-induced damage.

When we think about requirements for appurtenant structures such as spillways and outlets, the statement (1) can be paraphrased into the following statement:

A dam is required (3) not to release uncontrolled outflow discharge from reservoir during as well as after the L2 earthquake motion.

CASE STUDIES ON EARTHQUAKE DAMAGE TO DAMS

The Chi-chi Earthquake, Taiwan (M7.5, September 21, 1999)

This earthquake caused devastating collapse of Shih-Kang dam, as shown in Fig. 1. The dam is a concrete gravity dam, constructed in 1977. The damage was mainly caused by the surface rupturing of the seismic fault crossing the dam body [2]. Apart from the collapsed portion, structural damage to the dam body that included concrete cracking and joint openings were apparently concentrated at structural discontinuities such as interface between the piers and spillways, and contraction and uplift joints. The concrete cracking developed in the area of the spillway No. 6 through No.11 was found to be 3-4 mm wide, 10-30 m long and at almost 45 degrees against the dam axis, implying excessive compression from both bank sides.

After repeated discussions from various viewpoints, it was officially decided that the dam would be used again after completing rehabilitation work. The work included consolidation grouting over the entire area of the dam body in order to improve water tightness of concrete and foundation rock damaged by the faulting. Micro-tremor measurement was conducted on the dam three times by the author. Due to limitation of pages, the first measurement only is briefly described here.

Fig. 1 Shih-kang dam damaged by the 1999 Chi-chi Earthquake

In Fig. 2, vibration components in three directions are shown with locations of each observation point indicated on the left end side. All the observation points except LA, RA and GD were selected on the dam crest just above the piers where the girders are simply supported. From Fig. 2, following findings were drawn.

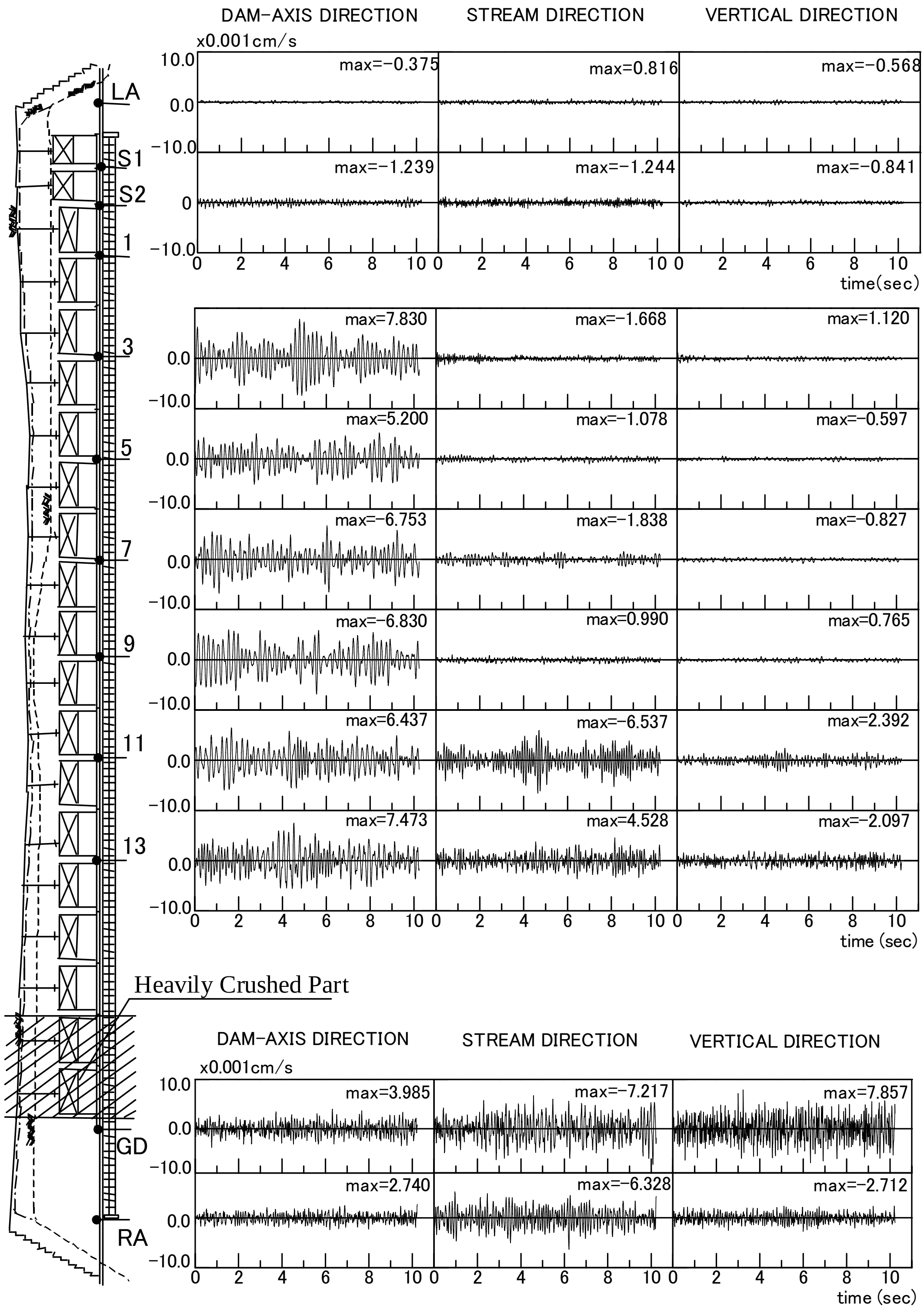


Fig. 2 Micro-tremors observed in October, 1999.

To the left side of the collapsed portion, vibration level of the dam-axis-direction component was relatively high at the point Nos. 3 through 13. The vibration component commonly showed a predominant period of about 0.2 sec, which was supposedly associated with the out-of-plane vibration of the cantilever piers.

Similarly to the left side, the vibration in both the stream and vertical directions was remarkably higher at the point Nos. 11 and 13 than the rest. Probably, this was due to some kind of serious damage widely developed over the right half side of the dam body such as extensive concrete cracking and separation between the dam and foundation rock.

To the right side of the collapsed portion, all the vibration level in the three directions was very high in contrast with that of the left side, which was attributable to the severe cracks in the dam concrete and disturbance caused by the water running through the cracks in torrents.

The measurement apparently suggested that the right half side of the dam body was severely damaged to the extent of fracturing of concrete and rock as a result of the seismic faulting, while the left half side suffered relatively less structural damage despite extensive cracks observed on the spillways, which was demonstrated later by the consolidation grouting.

The Niigata-ken Chuetsu Earthquake, Japan (M6.8, October 23, 2004)

Asagawara dam is a 37 m high zoned fill dam built in 1945, having upstream side slope of 1:3.0～1:2.8, and that of downstream of 1:2.5～1:2.0 [3]. During the main shock, longitudinal cracks and steps were formed along the crest, as shown in Fig. 3. The maximum difference of the steps was about 50 cm. After penetrating liquid plaster in the cracks, trenches were excavated to trace the cracking in the dam body, as shown on the right of Fig. 3. As a result, the cracks were found not to reach the clay core which remained almost free from earthquake damage. To restore the damage, all the loosened part near the crest was removed and re-banked to the original cross section with newly borrowed materials.

Fig. 3 Damage to Asagawara dam and trench investigation

Shin-yamamoto dam is 42.4m high, and has a crest length of 1,392m with a standard cross section shown in Fig. 4, built in 1990 [4]. This is a zoned fill dam with a dam axis of a semi-circular shape. Both right and left ends of the dam axis plunge into natural slopes. To facilitate the rapid draw down of the water level, there is a horizontal drain layer on the upstream side at the elevation of EL.143.8m~147.3m, which is just above the low water level.

Settlement of the crest was as large as 50~85cm on the left half side where the shelter zone is founded on terrace deposits. Due to the large settlement, a top of an H-shaped steel beam which was embedded to protect monitoring cables from bottom to crest in the filter zone was observed protruding about 30cm from the asphalt pavement as shown in Fig. 5.

On the upstream side near the left end, boiled sand was found at several places on the riprap surface at EL. 145～150m and nearby sedimentation as shown in Fig. 5. The origin of the boiled sand was mostly concentrated in and around the drain layer whose sand content was higher than that at the construction stage. In addition, near the left end of the dam, fine sedimentation was thick enough to cover the mouth of the drain layer. It seemed reasonable to think that the thick sedimentation in front of the drain layer had caused pore water pressure buildup under the strong shaking followed by the sand boiling.

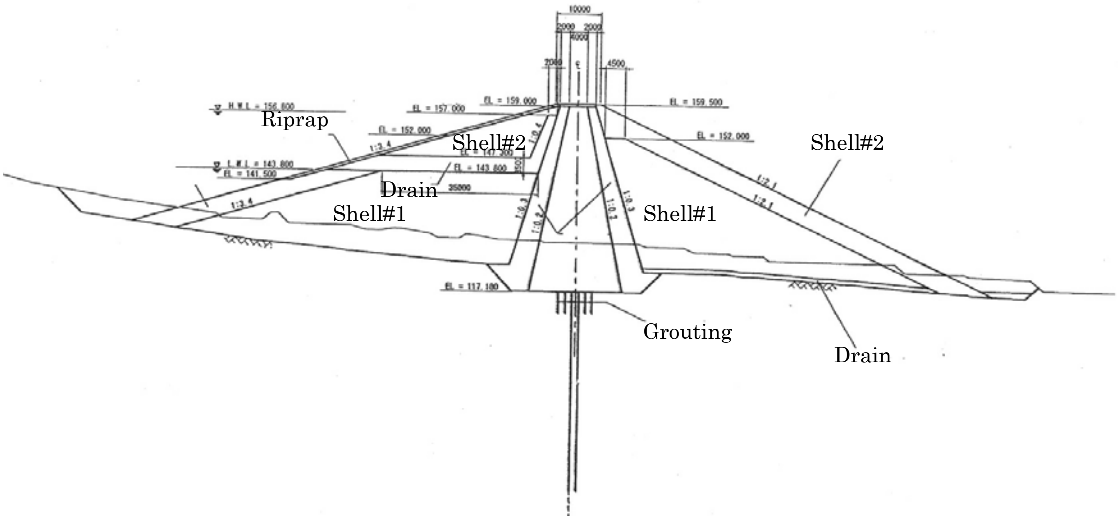


Fig. 4 Standard Cross Section of Shin-yamamoto Dam

Fig. 5 Settlement and Sand boils observed at Shin-yamamoto dam

Although strong motion observation was being conducted at the dam, it was regrettable that the main shock records were deleted by over-writing repeated aftershock records on them.

For months, causes and effects of the damage to the dam were closely investigated, and the damage was recognized as fairly limited and repairable. As this dam is located in a most snowy area in Japan, almost all the investigation and repair work at the dam was finished by the beginning of the winter season of 2005. The majority of the repair work was removal of the drain and shell #2 materials contaminated with boiled sand and sedimentation to a certain depth of the left half side of the dam, which was followed by re-embankment with sound materials and careful compaction. In addition, in order to prevent the sedimentation from accumulating in front of the drain layer of the dam, inflow training walls were constructed in the reservoir.

In view of the aforementioned statements for the safety requirements, the dam withstood the L2 motion in the near field of the 2004 earthquake somehow or other, and satisfied the requirements (1)～(3) despite considerable damage to the dam body. The basic concept of the requirements is clean-cut and seemingly acceptable not only to dam engineers but also general citizens.

The Iwate-Miyagi inland Earthquake, Japan (M7.2, June 14, 2008)

Aratozawa dam shown in Fig.6 is a 74.4 m high rockfill dam with a central clay core, built in 1998 and located at 15km south of the epicenter of this earthquake [5]. The strong shaking of the main shock caused a large amount of slope failure in the upstream of the dam with an estimated volume of about 67 million m^3, as seen in Fig. 6.

In a vertical section of the middle part of the dam, 3-component accelerometers are installed at the dam crest, mid-core and bottom gallery. Strong-motion acceleration was well recorded not only during the main shock but also the aftershocks and before the earthquake. During the main shock, the acceleration exceeded 1,000 gals at the gallery, as shown in Table 1 and Fig. 7. Despite such high acceleration, the dam body escaped serious damage, only suffering large settlement of the dam body. Based on these records, earthquake response of the dam was analyzed, especially from a viewpoint of non-linear response.

Fig. 6 Large-scale Landslides in the Upstream of Aratozawa dam

The analysis showed that the fundamental period of the dam was about 0.4s before the earthquake, and it was elongated to 1.2s and shortened again to 0.5s during the main shock. The change in the vibration period was attributed to the change of the wave velocity in the dam core, from 520m/s before the quake and the reduction to 200m/s and recovery to 380m/s during the main shock. The strong shaking induced large shear strains in excess of 10^{-3}. Due to the large strain, the shear modulus G showed a remarkable decrease from the initial shear modulus G_0. As a result of the decrease in G, wave velocity was reduced and the vibration period of the dam was elongated. Toward the end of the main shock, the modulus G in the core showed a gradual increase, but below G_0 and was anisotropic.

Table 1 Peak Acceleration observed at Aratozawa dam

	Stream dir.	Dam-axis dir.	Vertical dir.
Top (dam crest)	525gal	455gal	622gal
Middle (mid-core)	536gal	478gal	470gal
Base (gallery)	1,024gal	899gal	691gal

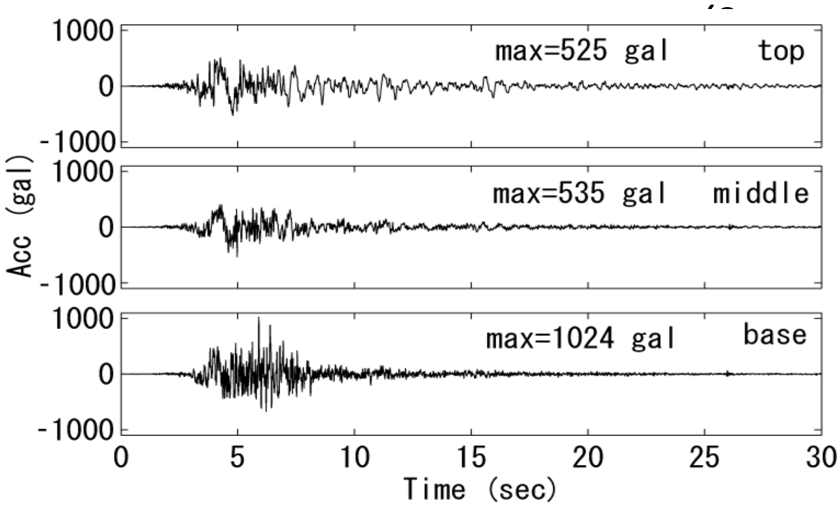


Fig. 7 Accelerogram in the Stream Direction observed at Aratozawa dam

Fig.8 Cracks and Trench Investigation at Izawa dam under Construction

Izawa Dam which will be a 132m-high rockfill dam after completion, was strongly shaken at the middle of the construction work. Many cracks were seen on the surface of the core and filter zones, and trenches were excavated to investigate features of the cracks as shown in Fig. 8. The cracks were 0.2-0.5m deep in the core, and 0.7-2.4m deep in the filter, respectively.

The 2008 Wenchuan Earthquake, China (M8.0, May 12, 2008)

There are various dam types in the earthquake-stricken area. Reportedly, there was no single large dam breached in the area and no secondary disasters happened due to the safety of dams. However, strong shaking of the earthquake brought damage to some dams and appurtenant structures.

Zipingpu dam is a 156m-high concrete faced rockfill dam (CFRD), with a storage capacity of 1.11×10^9 m^3. The dam crest is at EL.884m, having total length of 664m and width of 12m. The upstream slope is 1:1.4, and the downstream slope is 1:1.5 above EL.840m and 1:1.4 below that. The water level was at EL.830m at the time of the main shock. In the powerhouse, 4 power generators (190 MW each) are installed with a total capacity of 760 MW. Water impoundment was started in December, 2004.

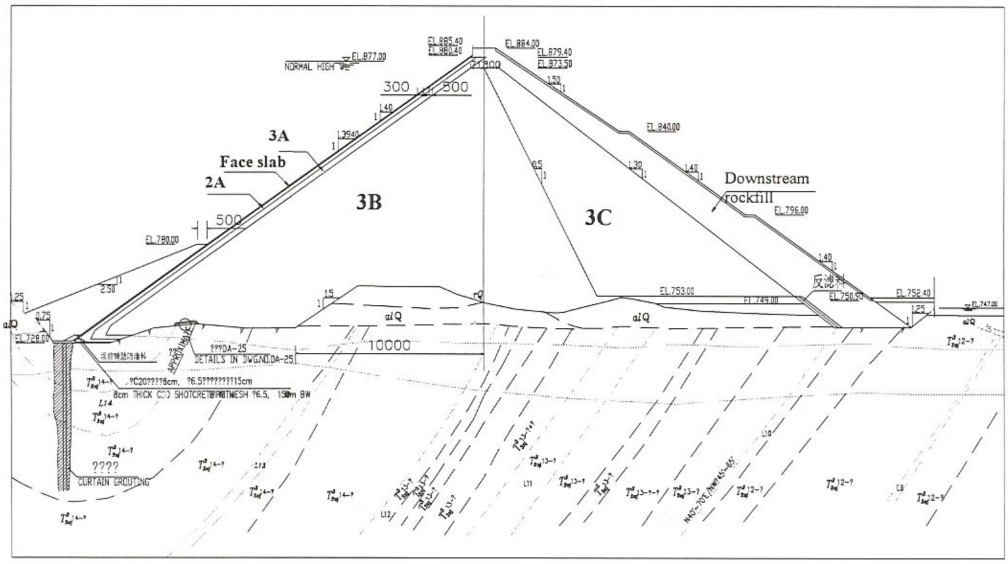


Fig. 9 Typical Cross Section of Zipingpu dam [6]

The basic earthquake intensity of the main shock was IX～X at the dam site, which exceeded the design earthquake intensity VIII with the peak ground acceleration (PGA) of 0.26 g. The distance from Zipingpu CFRD to the epicenter was 17 km, and the shortest distance from the fault was about 8km. Among strong motion accelerometers installed at 6 locations on the dam, main shock records were recovered from 3 installations near the crest. The measured PGA was about 2.0 g. Despite the high acceleration, the Zipingpu dam remained structurally stable and safe. The main features of the earthquake damage were displacement such as maximum settlement of 74 cm on the crest of the dam, horizontal displacement of 18 cm to the downstream, and 1 cm along the dam axis (from both sides to the river center). Other damage includes horizontal dislocation and overlapping of the concrete slabs on the upstream surface, which was observed at the construction joint at EL.854 m between the second and third stages. The leakage of the dam was 10 and 19 l/s, before and immediately after the earthquake, respectively.

Although the rockfill surface near the top of the downstream was cemented, it was partly loosened and displaced, especially at around the middle section of the dam, where separation of concrete side blocks from the concrete pavement and crushing of the concrete parapet wall were also observed as shown in Fig. 10, suggesting a large earthquake response. On the downstream crest near both bank sides, stone banisters were cracked down. The powerhouse and control building at the downstream toe of the dam were practically free from severe damage as shown Fig. 11, except settlement of the ground. The power generation was resumed 5 days after the main shock.

Fig. 10 Damage to the crest of Zipingpu dam

Fig. 11 Damage to the intake structure of Zipingpu dam

Shapai dam is a 130m-high roller-compacted-concrete (RCC) arch dam. Its crest arch length is 258m. The river valley is basically symmetrically V-shaped with a width-to-height ratio of about 1.7. It is known as the world highest RCC arch dam, having a bottom width of 28 m, and a crest width of 9.5 m. The crest elevation is EL.1867.5 m. The dam has only 4 joints, galleries at 3 levels and an elevator shaft. The dam construction was completed in May, 2002 and water impoundment was started in May, 2003.

As shown in Fig. 12, there are two spillway tunnels on the right bank. The bottom elevation at the intake tunnel No.1 is EL.1846 m with a design discharge of 244 m^3/s, and that of No.2 used to empty the reservoir is EL.1820 m with a design discharge of 197 m^3/s. A pressure diversion tunnel running across the mountain of the right bank is 3.49 km long and 3 m in diameter. The penstock has an inside diameter of 2 m, and a total length of 508 m. The designed earthquake intensity was VII, with PGA of 0.138g. The distance between the earthquake epicenter and Shapai RCC dam was 38 km and distance from the fault was 30 km. There was no seismometer installed on the dam.

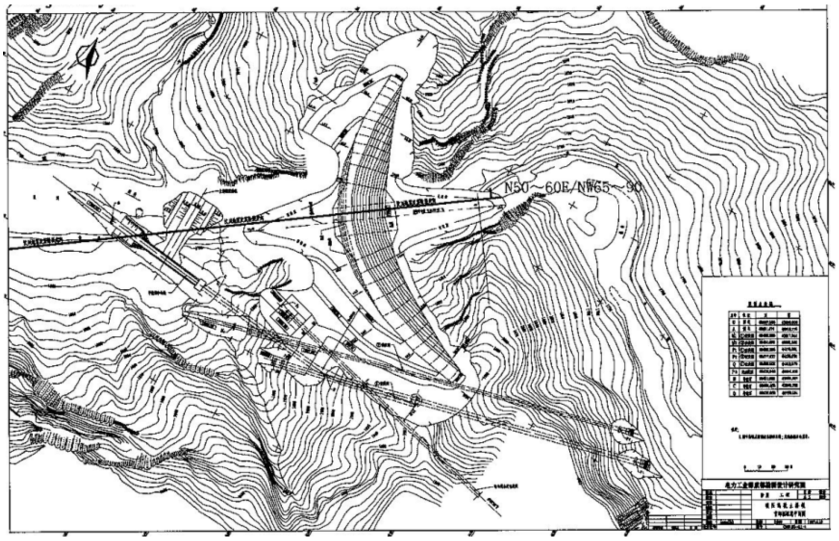


Fig. 12　Layout of the Shapai Hydoropower Project [7]

Fig. 13　Downstream and Upstream Sides of Shapai RCC Dam

Figure 14 shows the power house (left) with a total installed capacity of 36MW, and the switch yard (right). Due to intense impact of large falling rocks from both steep mountain slopes, severe damage was caused to side walls of the power house, the switch yard and the penstock. As a result, all the facilities in and around the power house, such as the control house, switch yard and generators were inundated by the water that mainly gushed out from the broken penstock, as shown with muddy gray colors in Fig. 14. Rehabilitation of the power house was to be finished in 2009.

Fig. 14　Shapai Powerhouse and Switchyard

Futang dam is a concrete dam, located on the main stream of Minjian river in Wenchuan city. Its major function is power generation with a total installed capacity of 360MW. The power house is on the left bank of the river. The dam is 31 m high, and 189.5 m along the crest. The normal water level is EL.1,268 m, and the storage capacity is 2.97×106 m^3. The water diversion system and the powerhouse system are on the left bank, including the intake, the diversion tunnel, the surge tank and penstock.

Earthquake-induced damage to Futang dam looked focused on the right bank side. As shown in Fig. 15, a concrete pier was attacked by a large amount of falling rocks from the steep slope of the right abutment. Due to the strong impacts of the falling rocks, a concrete retaining wall was fallen sideways and a tail water pier was cracked. Despite severe damage to these structures on the right bank side, facilities on the left bank side were little damaged.

Fig. 15 Futang Dam damaged by Rockfalls

SUMMARY AND CONCLUSIONS

From aforementioned case studies, we have learned that, in recent 10 years, earthquakes that occurred in the vicinity of dams caused various types of damage to dams. The damage was different from dam to dam, maybe due to the differences in the fault rupturing, strong motion intensity, dam configuration, construction practice, dam materials, and so on. These facts give us good lessons that some dams in the earthquake prone countries or regions should be reinforced to secure safety against large earthquakes as soon as possible. In this context, the validity and usefulness of the Guidelines [1] has been already demonstrated by a series of earthquake damage. Consequently, the next thing to be done is to apply the Guideline to as many dams as possible, or at least to all dams in Japan, and reinforce them adequately, if necessary, before a next big earthquake takes place.

REFERENCES

[1] Ministry of Land, Infrastructure and Transport. 2003. “Guidelines for Seismic Safety Evaluation of Dams Subjected to Level 2 Earthquake Motions (Draft)”

[2] Tatsuo Ohmachi. 2000.”On Damage to Dams in Taiwan due to the 1999 Chi-Chi, Earthquake”, Journal of Japan Society of Dam Engineers, Vol. 10, No. 2, pp.138-150.(in Japanese)

[3] Satoshi Takisawa, Tetsuo Shimamine, Shinichiro Nozawa, Tatsuo Ohmachi. 2007.”Damage of Asagawara Reservoir Dam Caused by the 2004 Niigataken-Chuetsu Earthquake and Consideration of

the Cause, Doboku Gakkai Ronbunshu C, JSCE, Vol. 63, No. 2, pp. 612-623, 2007. (in Japanese)
[4] Satoshi Takisawa, Tetsuo Shimamine, Shinichiro Nozawa, Tatsuo Ohmachi, Features and Causes of Damage to the Shin-Yamamoto Reservoir Dam by the 2004 Niigataken-Chuetsu Earthquake, Doboku Gakkai Ronbunshu C, JSCE, Vol. 64, No. 3, pp. 532-543, 2008. (in Japanese)
[5] Kazuhito SHIMAMOTO, Tatsuo OHMACHI. Hideaki KAWASAKI, Shinji IWAI. 2008.”Damage to Dams caused by the 2008 Iwate/Miyagi inland earthquake (Quick Report) Journal of Japan Society of Dam Engineers, Vol. 18, No. 1, pp.4-9. (in Japanese).
[6] Xu Zeping. 2009. “Performance of Zipingpu CFRD during the strong earthquake.” Reports and References, International Seminar on Earthquake and Dam Safety, 55-62.
[7] Chendu Hydropower Investigation Design & Research Institute (CHIDI). 2009. “The damage of Shapai, Futang, Yingsiuwan hydropower projects in 5.12 Wenchuan earthquake,” Hand-out distributed in International Seminar on Earthquake and Dam Safety.

A Sensor Network for Real-Time Damage Location and Assessment

Masanobu Shinozuka[1], Sehwan Kim[2], Pai H. Chou[3], Lu Fei[4], and Hong Rok Kim[5]

ABSTRACT

The unique feature of this study is to use accelerometers noninvasively installed on the pipe surface corresponding to the location where we would otherwise install pressure sensors invasively. We then take advantage of the experimental observation that a sharp change in the pressure is always accompanied by a sharp change in the acceleration on the pipe surface at the corresponding location along the pipe. This will make the entire procedure of water pressure monitoring into acceleration monitoring on pipe surface where the latter is significantly less costly compared with the former due to the fact that the acceleration measurement requires noninvasive sensing using generally much less expensive MEMS or other acceleration sensors rather than expensive pressure gages in an invasive mode for pressure monitoring. Thus, monitoring is made not for MWHGr (Maximum Water Head Gradient) but for MPAG (Maximum Pipe Acceleration Gradient). As a first step, using a small scale pipe network, this paper demonstrates a result of an field experiment that serves as the proof of concept of this new technology which represents a prototype of the next generation of SCADA for water distribution systems.

INTRODUCTION

Urban water delivery network systems, particularly the underground pipeline networks, can be damaged due to earthquake, pipe corrosion, severely cold weather, heavy traffic load on the ground surface, and many other man-made or natural hazards. In all these situations, the damage can be disastrous: interruption of potable water supply will create major human health problems let alone all kind of inconveniences; pipe damage may result in reduction in the water head diminishing post-earthquake firefighting capability; water leakage at high pressure may threaten the safety of near-by buildings due to scouring of their foundations; flooding could create major traffic congestion if pipe ruptures under a busy street. Yet, the current technology is not capable of accurately identifying the location and extent of the damage easily and quickly, especially immediately after a major earthquake. This paper demonstrates potential use of a sensor network for identification of location and extent of damage in real-time so that emergency response measures can be rapidly implemented to minimize disaster consequences.

[1] Distinguished professor and chair, Department of Civil and Environmental Engineering, University of California, Irvine, CA 92612, USA, E-Mail: shino@uci.edu
[2] Graduate researcher, Department of Electrical Engineering and Computer Science, University of California, Irvine, CA 92612, USA, E-mail: shawn.kim@uci.edu
[3] Assistant professor, Department of Electrical Engineering and Computer Science, University of California, Irvine, CA 92612, USA, E-Mail: phchou@uci.edu
[4] Visiting scholar at UCI, College of Civil Engineering, Southeast University, China, E-Mail: luifei@seu.edu.cn
[5] Visiting scholar at UCI, Center of Embedded Software Technology, Korea, E-Mail: hrkim@cest.re.kr

More specifically, this paper develops and demonstrates an advanced sensor network for real-time monitoring and condition assessment of utility water distribution systems particularly during and after a severe earthquake. The sensor network consists of a platform of multiple real-time wireless and energy-efficient sensors and sensor nodes. Each node transmits wirelessly the data sampled by Micro-Electro-Mechanical Systems (MEMS) and other emerging sensors. Collectively, these sensors have been assembled into a package called DuraNode or its miniature version Eco. Both are designed, assembled, and tested in the laboratory as well as in the field at UC Irvine particularly being tailored for civil engineering applications. Especially, the current generation of DuraNode and Eco are equipped with tri-axial MEMS accelerometers. The accuracy of these devices has been verified against traditional high-precision piezo-electric accelerometers in the field and by shaking table tests. Indeed, these tests have validated the ability of the sensor to make the low-frequency observations necessary for monitoring and remotely visualizing (by wireless communications) large-size civil engineering structures in real-time. The substantial cost-effectiveness, robustness, durability, small size and light weight of DuraNode and Eco sensors make densely configured observational networks possible for many types of civil infrastructure systems such as bridges, buildings, and pipelines networks.

However, here in this study we concentrate on its application to pressurized water distribution systems, and develop methods of rapidly detecting and locating the source of anomalies in the water system. Such anomalies can be caused by one of many events such as a pipe rupture and pump failure. There are two major hydraulic behaviors we take advantage of in order to develop a novel means of identifying the location and extent of pipe damage.

First, the numerical simulation using transient hydrodynamic analysis software shows that the temporal pressure change is larger at a location closer to the source of transient and decays with distance in both pipe directions. Therefore, if we install populated pressure sensors throughout the water network, say at each network joint, and continuously monitor the pressure there, the damage location(s) can be identified in the pipe between the two adjacent joints where the local maximum water head gradient (MWHG) are observed simultaneously. Additional study is needed to confirm a reliable correlation between the extent of the damage and maximum MWHG values. Second, experiments show that a sharp change in the water pressure is always accompanied by a sharp change in the acceleration on the pipe surface at the corresponding location along the pipe. This makes it possible to replace the entire process of water pressure monitoring with acceleration monitoring on pipe surface where the latter is significantly less costly compared with the former due to the fact that the acceleration measurement requires noninvasive sensing using generally much less expensive MEMS acceleration sensors rather than expensive pressure gages in an invasive mode for pressure monitoring.

Thus, monitoring is made not for MWHG but for MPAG (maximum pipe acceleration gradient). As a first step, using a small-scale pipe network, this paper shows the result of a field experiment that serves as the proof of concept of this new technology, which represents a prototype of the next-generation of SCADA (Supervisory Control And Data Acquisition) for water distribution systems.

DAMAGE DETECTION AND LOCALIZATION OF PIPE NETWORK

Hydraulic Transients

A hydraulic transient represents a temporary, often violent, change in flow pressure, and other hydraulic conditions in a water delivery system from an original (first) steady state to a final (second) steady state the system achieves after the effect of the disturbance that caused such a transient is absorbed into the second state. The disturbance includes such events as a valve closure or opening, a pump stopping or restarting, and pipe damage or rupture leading to substantial water leakage. The transient can produce a significant change in the water head and pipe pressure. In fact, as described in more detail in what follows, it is envisioned that the sudden change of such pressure will generate a measurable pressure wave and can be used for detection and localization of pipe damage. If the magnitude of this transient pressure is beyond the resistant capacity of system components, it can induce a significant damage on the pipes possibly resulting in equally significant system failures. Therefore, it is important to simulate the transient behavior of the water system under various adverse scenarios in order to understand the magnitude of these effects. In this study, the industry-grade computer code HAMMER (Haestad 2003) is employed to generate time histories of key hydraulic parameters (primarily water head and flow rate). The analysis is carried out for a hydraulic system as shown in Figure 1 that appears in HAMMER User's Guide. This water system consists of two reservoirs, one pump, one valve, 38 nodes and 54 pipe links. In the following, we consider first a case in which a pipe-rupture occurs at the mid point of link 111. In this case, a new node is created at the rupture location in this link (double circle in Figure 1), and the numerical analysis continues. The time history of the computed water head at Joints 9 and 11are plotted respectively in Figure 2 (a) and 2(b). Secondly, we consider a sudden stop of the pump station (node PMP1) due, for example, to seismically induced power blackout. The corresponding water head transient behavior is quite dramatically time variant as shown in Figure 2 (c) and 2 (d) computed respectively at Joints 13 and 20. The water head transient behaviors under other pipe-damage scenarios with appropriate physical parameters of nodes and pipes are shown in Shinozuka and Dong, 2005a.

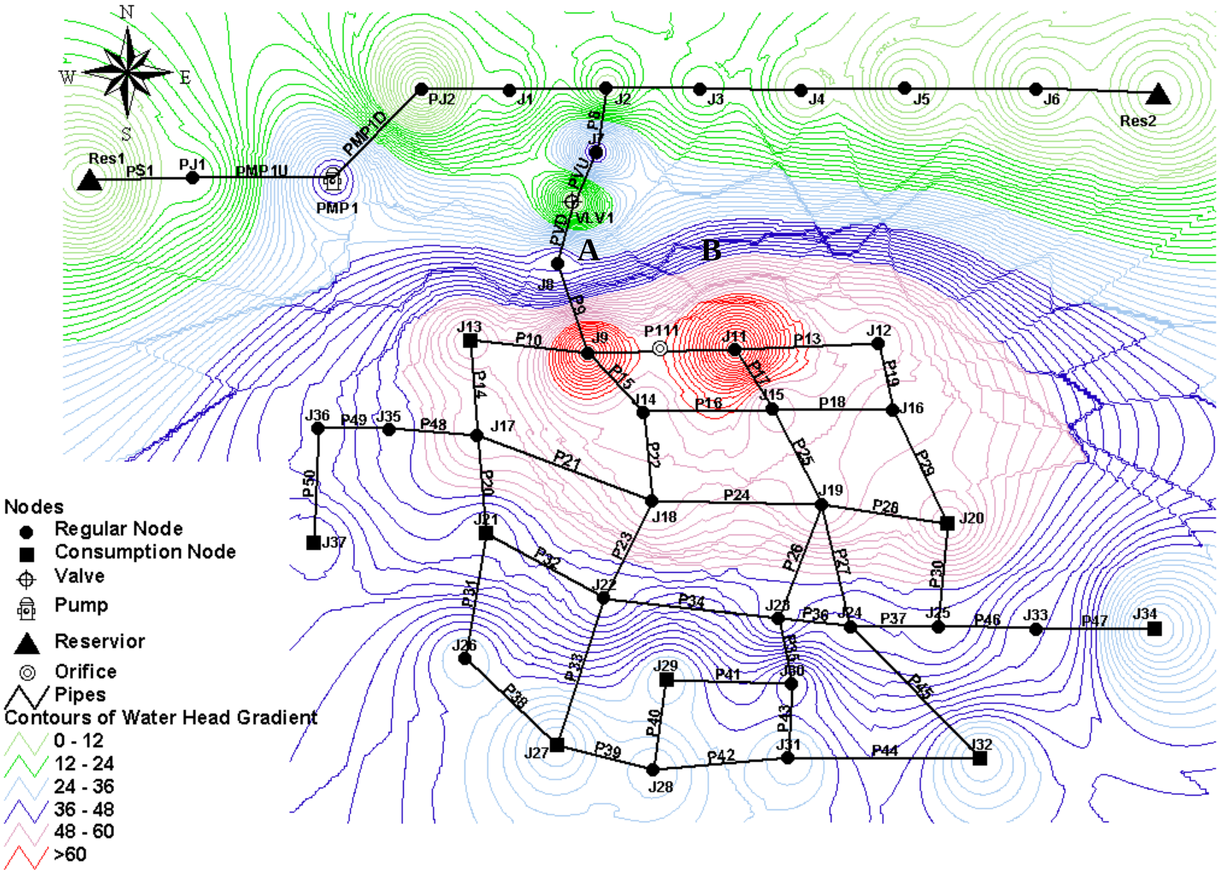
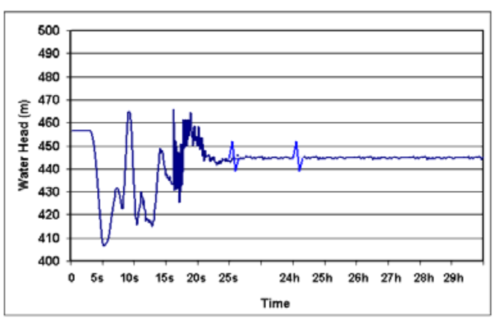
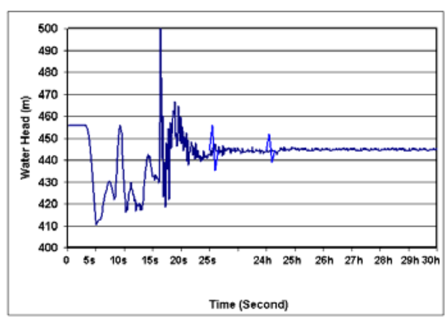


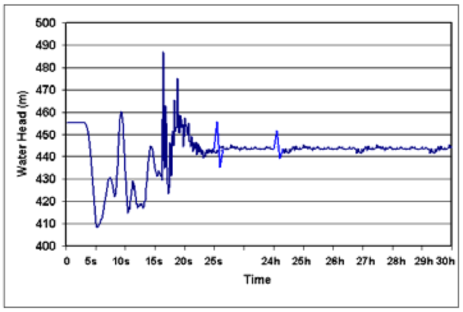
Figure 1. Distribution of water head gradient due to pipe P111 break



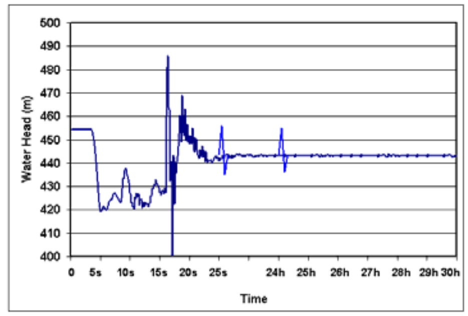
(a) transient water head at Joint 9 due to a rupture in pipe 111



(b) transient water head at Joint 11 due to a rupture in pipe 111



(c) transient water head at J13 due to pump stop



(d) transient water head at J20 due to pump stop

Figure 2. Nodal water head time histories under damage events

Damage Detection

A method of damage detection and localization, including the identification of malfunctioned equipment (typically pumps) is described here based on the comparison of the hydraulic parameters (water head in this case) before and after each damage event. For the primary purpose of a rapid detection and localization, it is most effective to catch the sign of abrupt change at the outset of the event. Fortunately for a sudden change such as a pipe rupture and pump stoppage, the response of the network is rapid particularly in the neighborhood of the source. This suggests that some measurable signature that indicates the rapidity of this change can be used for the purpose of such an identification. One convenient quantity that serves this purpose is the water head gradient as defined below.

$$D = \left| \frac{H_2 - H_1}{t_2 - t_1} \right| \tag{1}$$

Here H_2 and H_1 are the water head at a joint of interest at time t_2 and t_1, respectively. In this study. $t_2 – t_1$=0.2 (seconds) is used for computation.

During the normal steady state operation, D is usually negligibly small. In this paper, the water head gradient measured at the joints are integrated into a GIS platform for real-time visualization and for other advantages. Figure 1 shows the distribution of water head gradient D in a contour plot in the extended network space for the convenience of visualization. The contour plot indicates that the damage location can be identified to be in Pipe 111 between nodes J9 and J11 where the water head gradients are locally at their maxima.

PROPOSED METHOD OF DAMAGE DETECTION

The Novelty of the Method

In this section, we introduce a novel damage detection method based on a wireless MEMS-sensor network which monitors the pipe surface acceleration typically at each network joint in a non-invasive fashion and compute in real-time a measure of acceleration-change (for example, pipe acceleration gradient similar to Eq. 1). To be more specific, MEMS-sensors are installed at all the joints in the pipe network so that at least two end joints of every link of the network are monitored. When a rupture or significant leakage occurs in the network, the sudden change in the water pressure propagate through the net work and induces corresponding change in the acceleration of pipe vibration. This change in the pipe acceleration is measured and on the basis of these acceleration data, the pipe damage can be found in the pipe between the two end joints, where the acceleration gradient values form local maxima. This is parallel to the observation, as demonstrated in Figure 1 by analytical simulation, that the damaged pipe is found between two end joints where the water head gradient form local maxima. The procedure utilizing the non-invasive pipe surface acceleration measurement facilitates an extremely simple and cost-effective identification of damaged pipe. We note that the development of the exact correlation between the water pressure and the corresponding acceleration on the pipe surface needs further analytical study assisted by calibration on the basis of scaled model tests, and the field tests on a segment of some actual water systems such as Irvine Ranch Water District and Orange County Sanitation District. For the field test, we plan first to take advantage of scheduled events by the system owner/operator which include valve opening or closing and switching on and off the pumps. In this connection, we also caution that the acceleration change reflects not only the effect of pipe damage but also other effects including soil-fluid-structure interaction, particularly under earthquake conditions.

All wireless sensing applications require the collected data or detected event to be transmitted to a central office for the analysis and assessment. Wireless platforms can be roughly classified into three types: real-time monitoring, data logging, and event-detection. The first is required to send immediately after the event the measured data, while the other two cases aim to collect data for later statistical analysis. The proposed sensor technology provides a platform with near-real-time monitoring system for wireless data acquisition, transmission, processing, analysis and decision making. The challenges to design a real-time-monitoring system are fast communication links, fair and efficient media access control (MAC) protocols, and low-latency routing protocols. Earlier wireless sensor platforms such as Mica2, Telos, and Tmote have low data rate, on the order of 19.2kbps-250kbps. Such platforms are not adequate for real-time monitoring and scalable system. In our monitoring system, Eco nodes currently contain a transceiver with 1 Mbps of bandwidth, to be upgrade to 2Mbps in the next version. Our system is a much more cost-effective, scalable approach comparing with other platforms.

WIRELESS SENSOR NODES DESIGN

Eco platform is composed of one base station serving up to 50 Eco nodes to support the proposed real-time monitoring damage localization methodology. The components of the Eco platform are shown in Table 1. The Eco node is ultra-compact, low power, low cost, and suitable for dense deployment with short wireless range. It is equipped with one triaxial accelerometer, a small chip antenna, a temperature sensor, and a flex-PCB expansion port. We take advantage of their characteristics and deploy multiple Eco nodes at the joints of water delivery network to find the damage location. Base station and Eco node can communicate each other via the available 2.4GHz wireless radio. The base station can connect to the host PC via 10/100 Mbps wired

Ethernet interface. This combination is expected to make the proposed real-time monitoring methodology accurate and cost effective.

Ethernet Base Station

Figure 3 shows the picture of the Base station hardware. It consists of two modules: Free scale module and 2.4GHz RF module. The Free scale module has 10/100Mbps Tx Ethernet port, RS-232 port and 40 -pins I/O port for the connection of 2.4GHz RF module using SPI interface. One Base Station can aggregate data from multiple sensors, of up to 50 Eco nodes. To aggregate data from multiple Eco nodes, we develop low-complexity, high throughput multiple-access wireless protocol based on the concept of pulling protocols. A base station may pull autonomously or may be transparent to the host by passing commands and data through. Depending on commands, sensor nodes can send multiple replies for a single command.

Table I. The specification of Eco and Base station

	Eco	**Base Station**
Size (mm)	13 x 11 x 8	76.2 x 114.3 x 31.7
Sensor	Triaxial accelerometer ±3g	None
Power Consumption	Max. 100mW	4.5W
Max. Air Data Rate (bps)	1 Mbps	2 Mbps
Battery	40mAh Li-Polymer (3.7V)	DC 6V/2A
Wired Interface	Serial, SPI	10/100 base/T Ethernet
Wireless Interface	2.4GHz Shockburst	2.4GHz Enhanced
Radio Range (m)	10 ~ 20	10~20
Cost ($) @ 1000	30-50	100

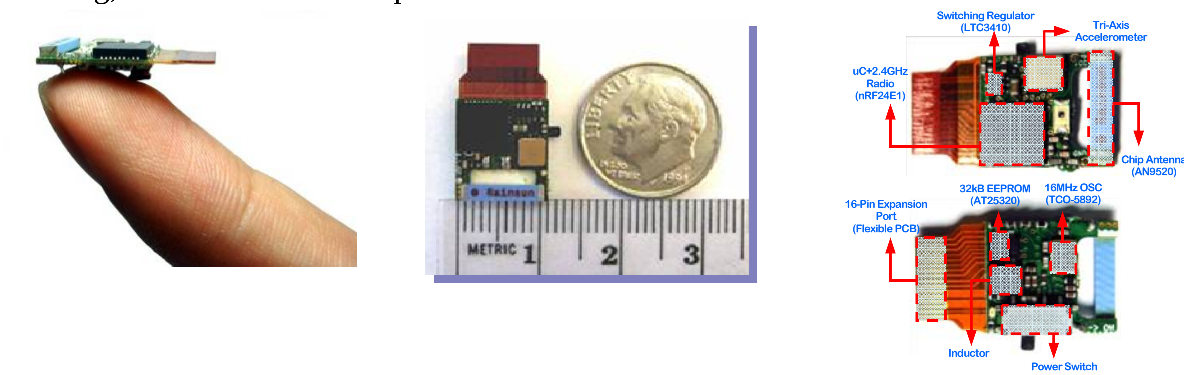
(a) 2.4GHz RF Module (b) Freescale Module

Figure 3. Photos of Base Station

Eco

The Eco is an ultra compact, low-power wireless sensor node. With the dimension of 13 x 11 x 7 mm including 40mAh Li-polymer battery, Eco is the world's smallest wireless sensor node to date, as shown in Figure 4. Also it consumes less than 100mW and transmits data up to 10m. The Eco node consists of five subsystems: MCU, radio, sensors, power, and expansion port. The MCU on the Eco node is the nRF24E1, which has an integrated 2.4GHz RF transceiver with a data rate of up to 1Mbps. The communication distance is up to ten meters. These features enable it to acquire data on a real-time basis. In addition, Eco has a triaxial acceleration sensor, the Hitachi-Metals H34C, with a ±3g range and temperature from 0-75°C. Eco also has an infrared light sensor

(S1087). Thanks to its ultra-compact size and low power consumption, Eco nodes can be applied to many kinds of scenarios, including medical diagnosis, environmental and structural-health monitoring, and new human-computer interface.



(a) On the finger (b) Size of Eco (c) Top/Bottom View

Figure 4. Photos of the Eco Sensor Node

PRELIMINARY EXPERIMENTS

Experimental Setup

To validate the concept for the multi-dimensional wireless transmission of high frequency acceleration data obtained from the MEMS sensors, a miniature water network model was constructed with 40 PVC pipes of 1.5-inch diameter and 2 valves labeled *A* and *B*. Figure 5 (a) shows the photo of this pipe network, while Figure 5(b) shows the overall size of this pipe network to be about 200 x 200 inch^2, where Valves *A* and *B* are used to control water pressure inside the pipe network and to emulate a rupture, respectively. Valve *A* can be adjusted manually to three states: closing, half-opening, and complete opening, where closing means high pressure and no water flow; half-opening means medium pressure with water flowing in the pipe network; and complete opening means low pressure with water flowing. The half-opening case is similar to real water delivery system with ambient noise.

Results and Analysis

Figure 6 shows the measured data of rapidly changed acceleration using an Eco-based MEMS sensor network. Each Eco node is equipped with a triaxal MEMS sensor, and 17 nodes with three channels each successfully transmitted acceleration at 25 samples per second in real time to a laptop computer. A sequence of X-direction acceleration records is plotted in Figure 6. Recording of the data begins when water pipe network reached steady state after injecting water to the pipe network, and recording stops after a few seconds when Valve *B* is forced to completely open abruptly to simulate a pipe rupture.

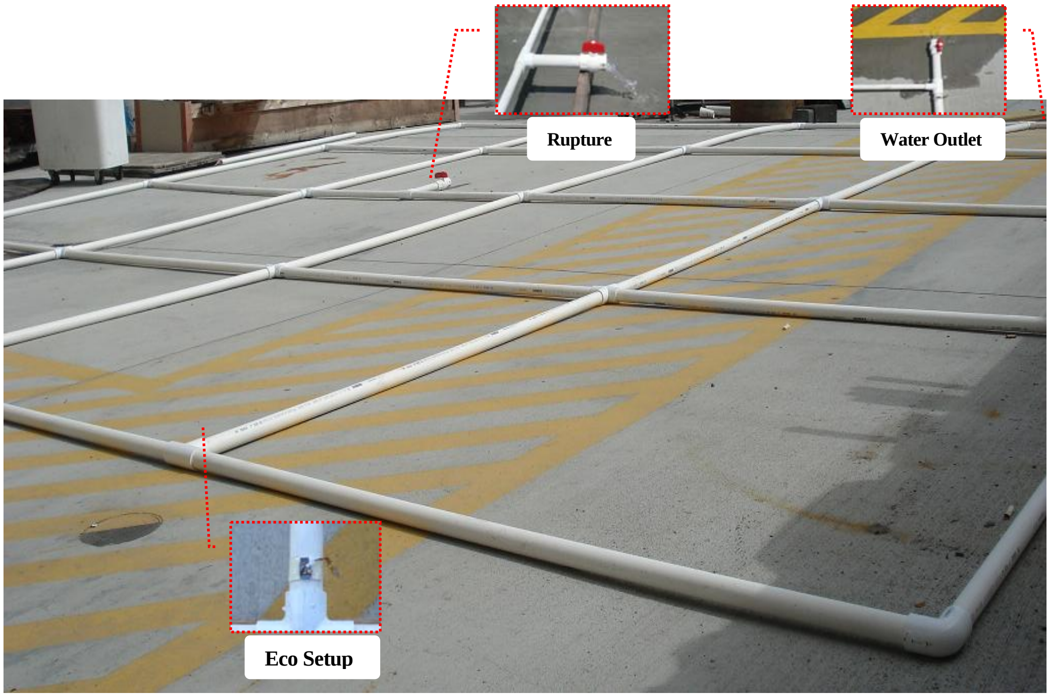
We plot representative acceleration data from eight of the 17 joints, and they are labeled 1, 3, 7, 8, 11, 13, 14, and 16. These plot show that the effect of simulated rupture measured in terms of the amplitude (intensity) of acceleration depends on the distance between the rupture location and the sensor locations. For example, Figure 6 (a), (b) show two representative acceleration data measured on the segment of rupture; (c), (d) show those measured one segment away from the rupture point; (e), (f) are those two segment away; and (g), (h) are those one and two diagonals away, respectively. The sharp change of acceleration in each chart corresponds to the event of opening Valve *B*.

Upon closer examination of Figure 6, we find that the amplitude of each peak is different. The amplitudes at joints 3 and 7 on the rupture segment are 2.75g and 2.2g, respectively. At one segment away (joints 8 & 13), they are 0.8g and 1.2g, and at two segments away (joints 1 & 16), they are 0.5g and 0.675g. This reveals that the acceleration change (which is almost equal to the acceleration itself because the ambient acceleration is negligibly small) is (locally) largest at the two ends of a segment which is the ruptured segment. The amplitude of the acceleration change decreases as one moves away from the rupture point in distance as shown in Figure 7 (a). Using these experimental results, we can plot a contour map for the convenience of visualization as shown in Figure 7 (b) which corresponds to the contour map in Figure 1. The simulated damage in this case is located in the inner most and smallest polygon. These experimental results confirm that the proposed method is promising in that the change in the pipe surface acceleration can be used as metric to develop the contour map from which the location and extent of pipe damage can be identified.

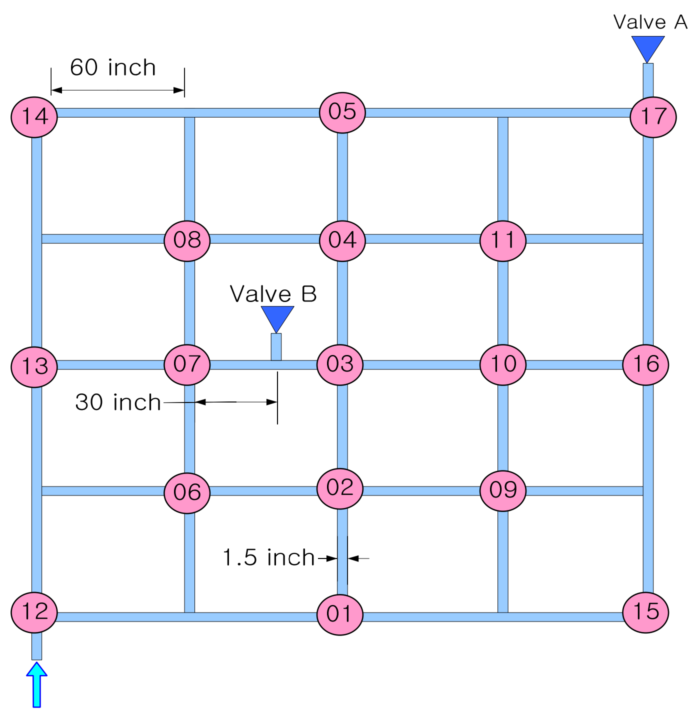
Initially, both valves are closed to allow the pressure to build up gradually, and then valve *A* was opened by half. This procedure can provide not only a semi-steady state water pressure inside the pipe network but also an ambient noise due to water flowing inside the pipe. Eco nodes are installed at 17 joint points on the water pipe network to collect vibration data in real-time, as shown in Figure 5(b). The data are wirelessly transmitted continuously to a host computer via a base station in near real-time, with about 1 second of lag.

CONCLUSION AND FUTURE WORK

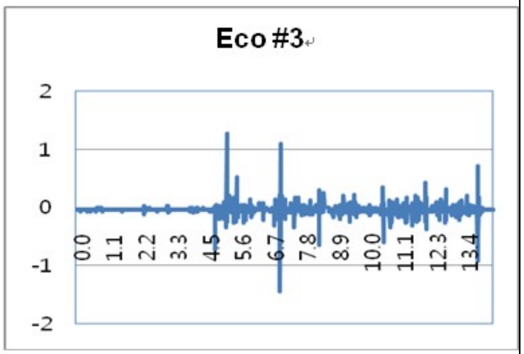
We propose a novel water-pipe damage detection method based on time-correlated acceleration data collected using a wireless MEMS-sensor network from different joints of a water pipe network. Each sensor measures the acceleration change on the pipe surface non-intrusively to determine rupture events and to locate the point of rupture. The results of preliminary experiment validate the concept of measurement of pipe acceleration for damage detection. To enhance the accuracy of detecting damage location in a larger-scale network, many improvements are needed, including more accurate time synchronization, improved wireless communication protocol, and better algorithms for data analysis, including the possible use of frequency-domain analysis. Further study is needed to correctly analyze the situations in sharp bends and T-joints and to understand the pipe vibration under the ambient and transient hydraulic conditions. We plan to install a new platform with greatly enhanced wireless communication capabilities on a subset of a regional water supply network such as the City of Westminster and the Irvine Ranch Water District where their existing SCADA measurements can be used for possible comparison.



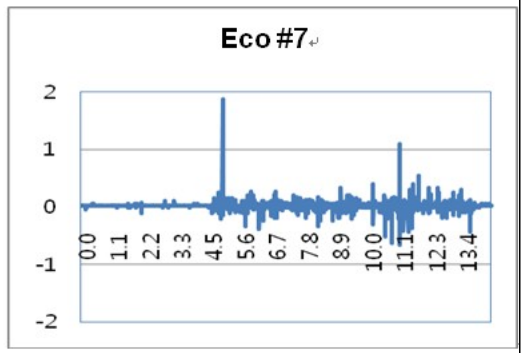
(a) Photo of a Small Scale Water Pipe Network



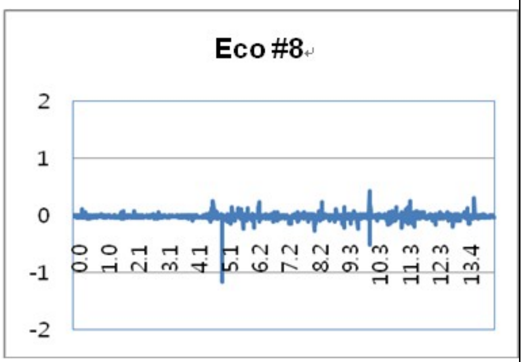
(b) Dimensions of the Model Water Pipe Network and the locations of 17 Eco sensors
Figure 5. Experimental Setup Showing a Small-ScaleWater Pipe Network



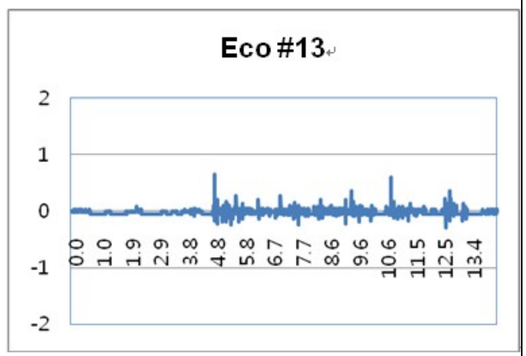
(a) Rupture segment (right)



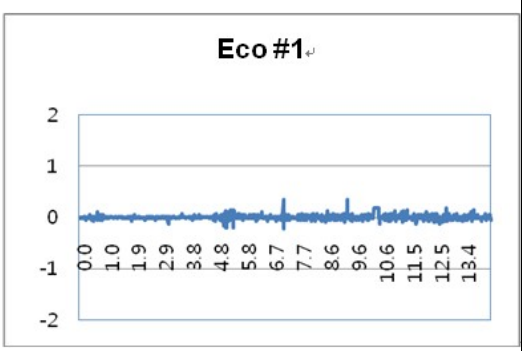
(b) Rupture segment (left)



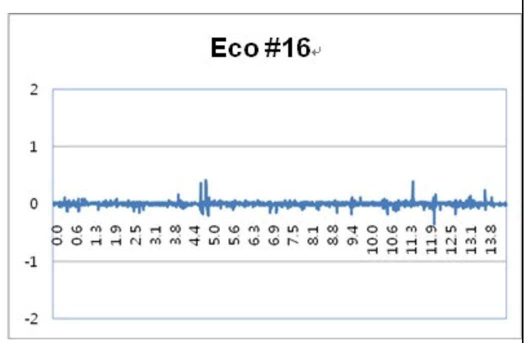
(d)One segment away (left)



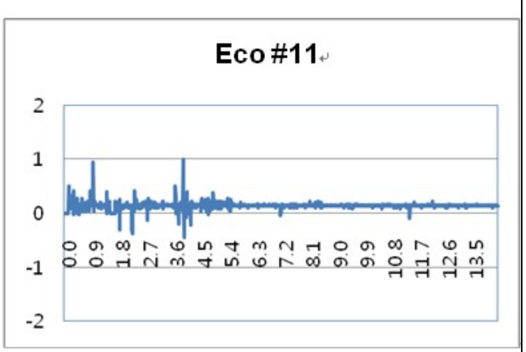
(c) One segment away (upper)



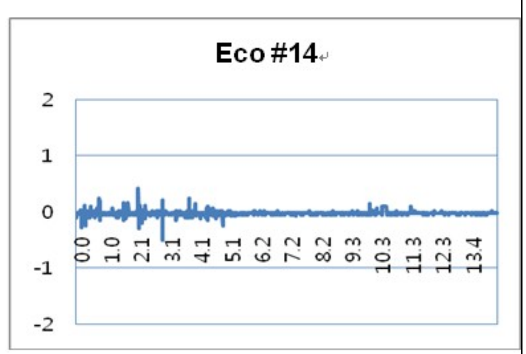
(d) two segment away (upper)



(e) two segment away (right)

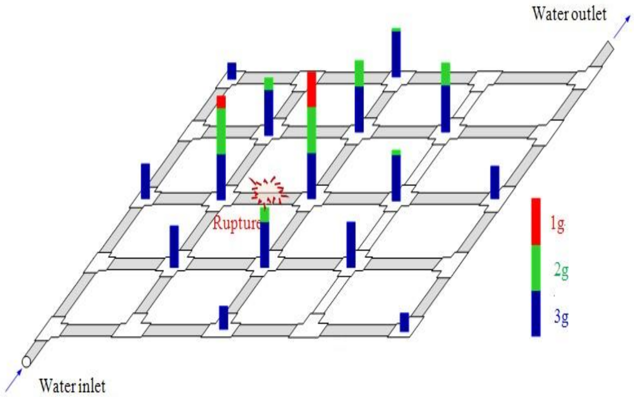


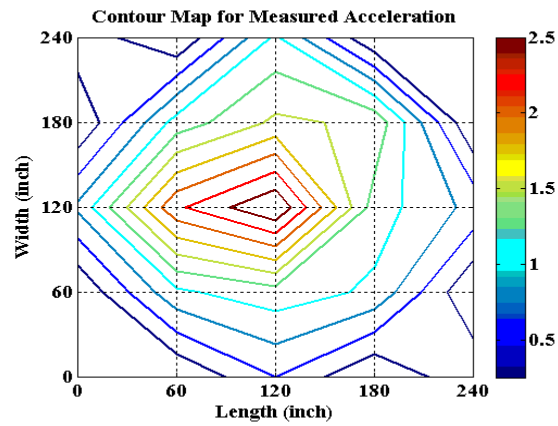
(f) one diagonal away



(g) two diagonal away

Figure 6. Acceleration Data measured by Eco nodes





(a) Visualized image for measure acceleration data (b) Contour Map drown by measured data
Figure7. The simulation results for a miniature water pipe network

ACKNOWLEDGMENTS

This study was done under National Institute of Standards and Technology Grant. Their support is immensely appreciated. The authors thank Eunbae Yoon and Changik Lee for their assistance in conducting many of the water pipe experiments. The authors also thank Chongjing Chen for providing the initial firmware for the Eco nodes and the base station.

REFERENCES

[1] Seung-mok Yoo, Chong-jing Chen, and Pai H. Chou,2009, "Low-Complexity, High-Throughput Multiple-Access Wireless Protocol for Body Area Networks," to appear, in *Proc. Body Sensor Networks*, June 3-5, Berkeley, CA, USA

[2] Chulsung Park and Pai H. Chou, 2006 "Eco: Ultra-Wearable and Expandable Wireless Sensor Platform," Proc. *Third International Workshop on Body Sensor Networks*, April 3-5. MIT Media Lab.

[3] Chulsung Park, Pai H. Chou, and Masanobu Shinozuka, 2005, "DuraNode: Wireless Networked Sensor for Structural Health Monitoring," to appear in *Proceedings of The 4th IEEE International Conference on Sensors*, Irvine, CA, Oct. 31 - Nov. 1

[4] Shinozuka, M. and Dong, X. 2005 a, "Evaluation of Hydraulic Transients and Damage Detection in Water System under a Disaster Event", *The 3rd US-Japan Workshop on Water System Seismic Practices*, Kobe, January 26-28

[5] Shinozuka, M. and Xuejiang Dong, 2005b,"Damage Detection and Localization for Water Delivery Systems" *Proceeding of the 5th International Workshop on Structural Health Monitoring*, Stanford University, Stanford, CA, September 12-14, pp. 1267-1273.

[6] Shinozuka, M., Park, C., Chou, P. and Fukuda, Y., 2006, "Real-Time damage localization by means of MEMS sensors and use of wireless data transmission," in *Proceedings of SPIE Conference on Smart Structures & Materials/NDE*, San Diego, CA, February 26 – March 2.

The Project of Reinforcement of Embankment for Measure Against Earthquake of the Murayama-Shimo Reservoir

Akira Suzuki[1] and Nobutaka Maruyama

ABSTRACT

The earth dam named the Murayama-Shimo Reservoir was completed 1927, which was the man-made lake, located on the hilly area called the Sayama-Kyuryo in the northwest of Tokyo. This reservoir, having a length of 587 m and a height of 33 m, has been the largest earth dam in Japan. This reservoir is exclusively for water supply in Tokyo, which becomes extremely important in supplying water at the time of disaster like earthquakes, power failure and so on, because of ability of sending raw water in gravity flow to several water purification plants on the downstream area.

Moreover, the area in front of the downstream slope of this reservoir has become a densely populated area.

After the Great Hanshin-Awaji earthquake (1995.1.17), we've performed the seismic response analysis for this reservoir. This analysis result concluded clearly that the embankment would be deformed and damaged by LEVEL-2 earthquake such as a 7 magnitude earthquake. And then, we decided the reinforcement works of the embankment in order to enhance the earthquake resistant ability.

This paper describes the design, structure and construction of a counter-weight fill with a 17 m-high geogrid-reinforced steep slope that was constructed on the downstream slope of this earth fill.

The seismic stability of the existing old earth dam was increased by means of the geogrid-reinforcement technology.

[1] Bureau of Waterworks, Tokyo Metropolitan Government

1 OUTLINE OF MURAYAMA-SHIMO RESERVOIR AND EINFORCEMENT

1.1 RESERVOIRS FOR DRINKING WATER IN TOKYO

Bureau of Waterworks, Tokyo Metropolitan Government, has three reservoirs on a hilly area, called Sayama-Kyuryo, in the northwest of Tokyo, which are Yamaguchi Reservoir (with a storage capacity of 19,530,000 m^3), Murayama-Shimo Reservoir (11,840,000 m^3) and Murayama-Kami Reservoir (2,980,000 m^3). They store the raw water transmitted from the Tamagawa River, running east of the site from the mountain area in the west of the metropolitan Tokyo. Their total storage capacity is about 35 million m^3, which is nearly the same as the amount of the water spent for one week in the 23 wards (i.e., the central urban area) of Tokyo. As they are located on a relatively high land, they can send the raw water in gravity flow to several purification plants existing in their downstream without using any artificial power. For this reason, they are extremely important in ensuring the water supply at the time of disaster like earthquakes, power failure and so on.

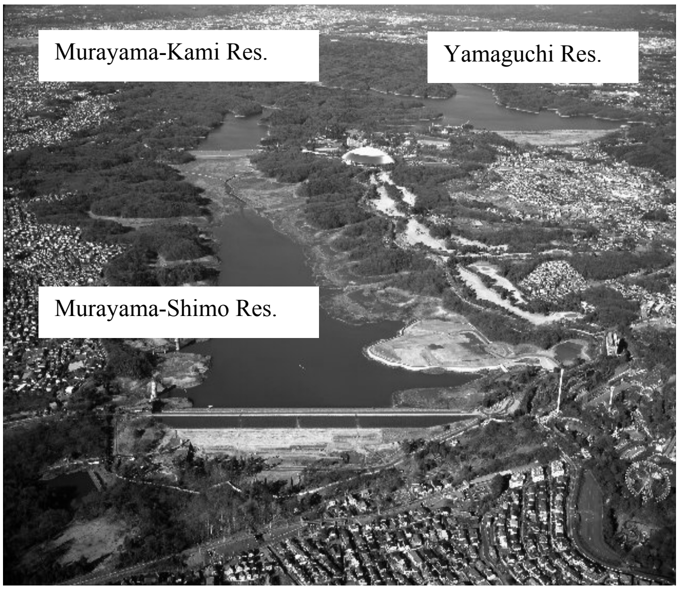


Photo.1. Air view of three reservoirs (the down-stream slope of the earth dam for Murayama-Shimo Reservoir under reconstruction)

1.2 MURAYAMA-SHIMO RESERVOIR

Murayama-Shimo Reservoir, of which reinforcement works are reported in this paper, was constructed exclusively for water supply use. The earth dam, which was completed on 1927, is a center-core type earth dam with a crest length of 587 m and a height of 32.6 m.

1.3 REINFORCEMENT WORKS FOR MURAYAMA-SHIMO RESERVOIR TO INCREASE THE SEISMIC STABILITY

After the Great Hanshin-Awaji Earthquake (1995), Tokyo Waterworks Bureau performed a seismic resistant analysis of these three earth dams and found that the crest of the dam would settle down about 1 meter if subjected to an intensive seismic motion, as those experienced during the 1995 Hanshin-Awaji Earthquake, caused by an earthquake having a magnitude of the order of eight. Also by taking into account the facts that the dam has become 80 years old and the immediate downstream of the dam has become a heavily populated residential area (as seen from Photo.1), it was decided to reinforce the earth dams for two relatively large reservoirs (Yamaguchi and Murayama-Shimo) in order to increase their seismic stability. The earth dam for the Yamaguchi Reservoir, which is larger than the Murayama Reservoir, was first reinforced by November 2002. Subsequently, the reinforcement works of the earth dam for the Murayama-Shimo Reservoir started May. 2004.

1.4 REINFORCING METHOD FOR THE EARTH DAM FOR MURAYAMA-SHIMO RESERVOIR

Tokyo Waterworks Bureau organized the technical advisory committee consisting of a number of leading specialists to determine of the reinforcing method. A geological survey and a geotechnical investigation at the site were performed and the internal structure of the earth dam was investigated. Based on the results from the above, the construction method, including the method to obtain the backfill material and the details of construction process were studied while the cost-effectiveness of several proposed methods was evaluated. The results of the investigation revealed that the stability of the existing earth dam when subjected to the considered high seismic design load may not be sufficient. To reinforce the earth dam, it was finally decided to arrange inclined and level drainage layers on the existing downstream slope of the earth dam to keep the groundwater level inside the dam sufficiently low and then construct a counter-weight fill above the drainage layers (Fig.1).

The counter-weight fill increases the confining pressure in the underlying soil layers in the existing earth dam , the natural ground, the drainage layers and thereby the shear strength of soil. By this way, the resisting moment against rotational failure in the downstream slope of the dam can be effectively increased. Furthermore, it was decided to cover the crest of the dam with a cement-stabilized soil layer to prevent the failure of the crest of the dam by a high seismic response.

The construction of the counter-weight fill was started September 2005 and completed June 2006. All the reinforcing works was completed March 2009.

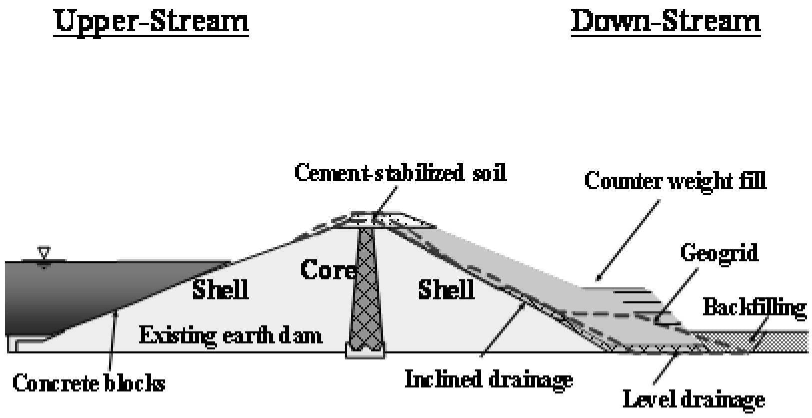


Fig.1. Cross-section of the earth dam for Murayama-Shimo Reservoir after reinforcement works

2 REINFORCEMENT WORKS OF THE EARTH DAM USING A GEOGRID

Despite that it was confirmed that the construction of a counter-weight fill on the downstream slope of the earth dam was the most cost-effective feasible method to reinforce the earth dam, the available space was highly restricted because the present area of the Sayama Park, existing adjacent to the downstream slope of the earth dam (Photo.1), should be reserved. To alleviate this problem, it was decided to make the slope of the lower part of the counter-weight fill relatively steep with a slope of 1 : 1 in H:V (i.e., the slope angle equal to 45 degrees) reinforced with polymer geogrid layers. The height of the steep slope of the counter-weight fill is about 17 m (see Fig.2).
The reinforcement of the high steep slope of the counter-weight fill with geogrid layers was decided taking into account an extremely high importance of its long-term stability for a long time of this relatively large scale earth structure. The construction of a counter-weight fill on the downstream slope of the dam was decided because a sufficiently high long-term stability of a geogrid-reinforced steep slope continuously and periodically under submerged conditions when constructed on the upstream slope could not be ensured. It was considered that, when constructed on the down stream slope, even in case the geogrid-reinforced slope is damaged, it does not directly affect the function of the reservoir while its repair works become quite feasible.

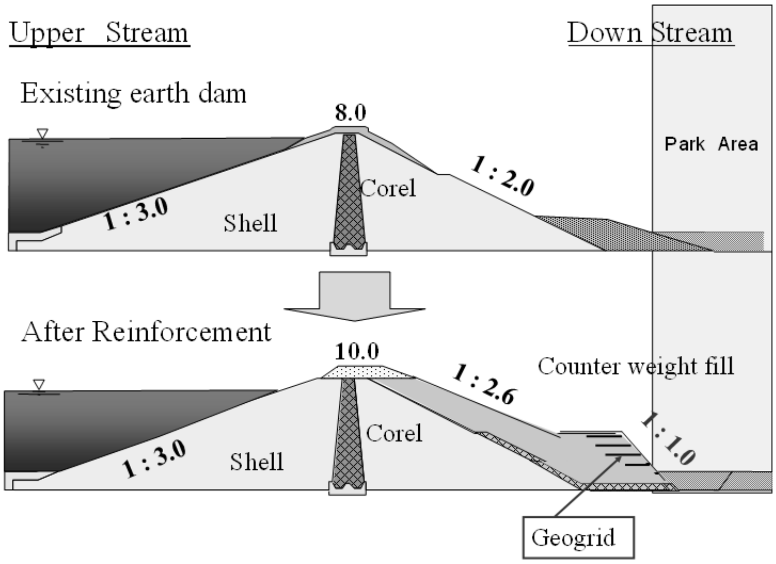


Fig.2. Reinforced earth fill dam using a geogrid

3 CONSTRUCTION MATERIAL AND THEIR PROPERTIES

3.1 BACKFILL MATERIAL

The backfill soil for the counter-weight fill, including a geogrid-reinforced slope, was obtained by mixing in a volume ratio of 1.0 : 1.0 : 1.0 on-site soil and a purchased gravel and a purchased sand. The on-site soil was obtained by excavating a small counter-weight fill that was existing at the toe of the downstream slope. Fig.3 shows the average grading curve of the backfill soil after mixing. The maximum size is 150 mm and the fines content ranges from 15 % to 30%. On a stockyard adjacent to the construction site, the excavated on-site soil was placed in a 20 cm thick layer, which was overlaid by another 20 cm thick layer each of purchased gravel and purchased sand respectively. This procedure was repeated. The backfill soil to be transported to the construction site was obtained by scraping these soil layers in such way that the three different types of soil are well mixed.

In the geogrid-reinforced slope, the backfill was compacted in a lift of 20 cm by passing a 10-ton class vibrating roller eight times. The minimum degree of compaction and minimum dry density to be satisfied were specified to 95 %, which were determined based on results from compaction tests in the laboratory (Fig.4) and confirmed by on-site compaction tests performed in advance. The molding water content was ensured to be within +2.0 % -1.5% from the optimum.

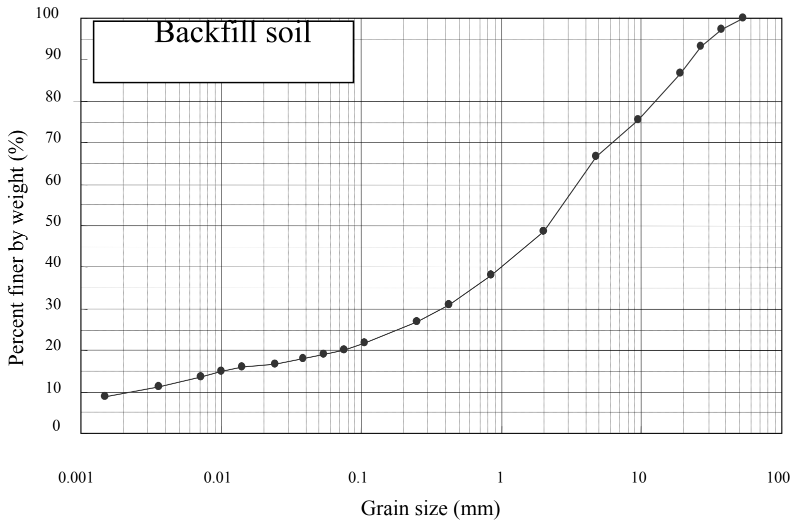


Fig.3. Example of grading curve of the backfill soil for the counter-weight fill

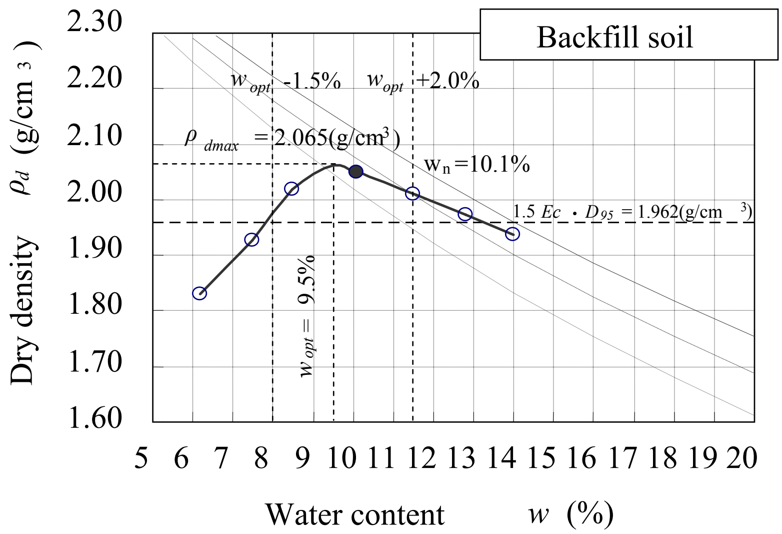


Fig.4. Example of compaction curve of the backfill soil

3.2 GEOGRID

A geogrid of high density polyethylene was selected to ensure a high interlocking with the backfill having some amount of fines. Four different types of geogrid having different rupture strengths were used depending on the required strengths at the respective zone in the steep slope. The nominal tensile rupture strengths obtained by tensile tests at a strain rate of 1% or 20%/min using 20 cm wide specimens were, respectively, 2.0 kN/m, 21.6 kN/m, 30.0 kN/m and 36 kN/m. It was investigated whether it is additionally necessary to arrange layers of a non-woven geotextile in the backfill to dissipate excess pore water that might build up during the construction of the counter-weight fill. It was found based on results from consolidation tests on the backfill material and associated analysis that this measures is not necessary.

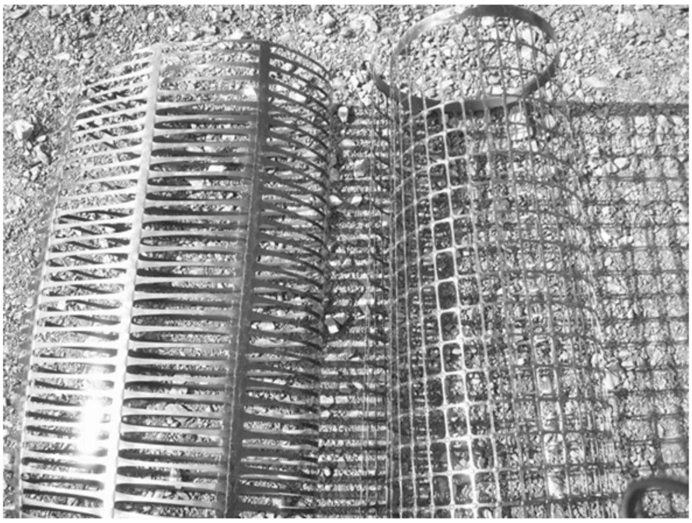
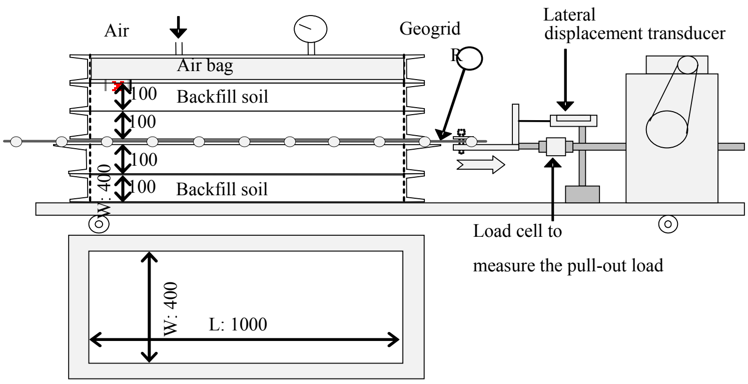


Photo.2. Geogrid for Primary (left) & Secondary (right) Reinforcement

3.3 PULL-OUT TESTS

Considering the fact that the backfill soil includes some amount of fines, a series of pull-out tests, as shown in Fig.5 and Photo.3, were performed. The pull-out displacement rate was 1 mm/min and the confining pressure applied ranged from 5 kPa to 70 kPa. It was confirmed that the bond shear strength at the interface between the backfill and a geogrid layer is sufficiently high; i.e., minimum bond shear strength expressed as: $\tau_f = c + \sigma\tan\varphi$ (c= 9.5 kPa; and φ= 26.4 degrees) could be confirmed.



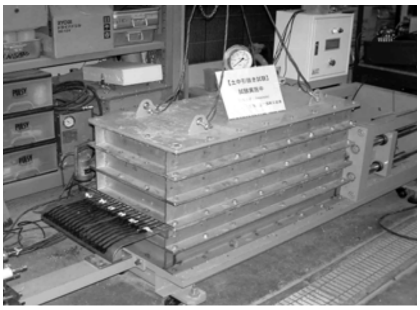


Fig.5. & Photo.3. Pullout tests of the geogrid used in the project (all the units in mm)

4 PLACEMENT OF GEOGRID LAYERS

4.1 STRUCTURE OF THE GEOGRID-REINFORCED FILL

Based on results of limit equilibrium-based stability analysis assuming circular failure planes, a cost-effective arrangement of geogrid layers that ensures the necessary seismic stability was determined. That is, eight 7 m-long layers of a geogrid having relatively high tensile rupture strengths 21.6 ~ 36.0 kN/m were placed with a vertical spacing of 1.6 m, as shown in Fig.6, photo.4, photo.6.
According to the necessary tensile rupture strength at the respective geogrid layer at a different level, four types of geogrids having different rupture strengths, described in the preceding section, were used as shown in Fig. 7.
Moreover, to prevent the surface failure in the steep slope, a number of 2 m-long low strength geogrid layers were arranged with a vertical spacing of 0.4 m, as show in Fig.6, photo.5, photo.6. The total area of the installed geogrid layers is 28,500 m^2.

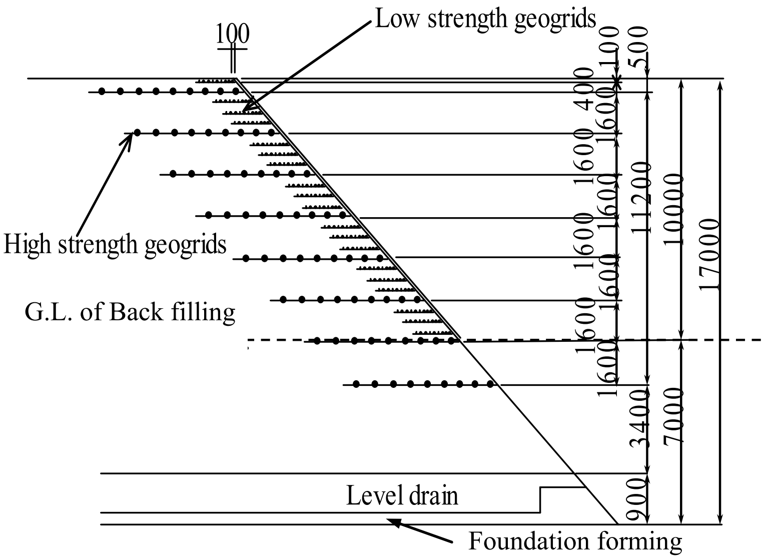


Fig.6. Cross-section of the geogrid-reinforced steep slope.

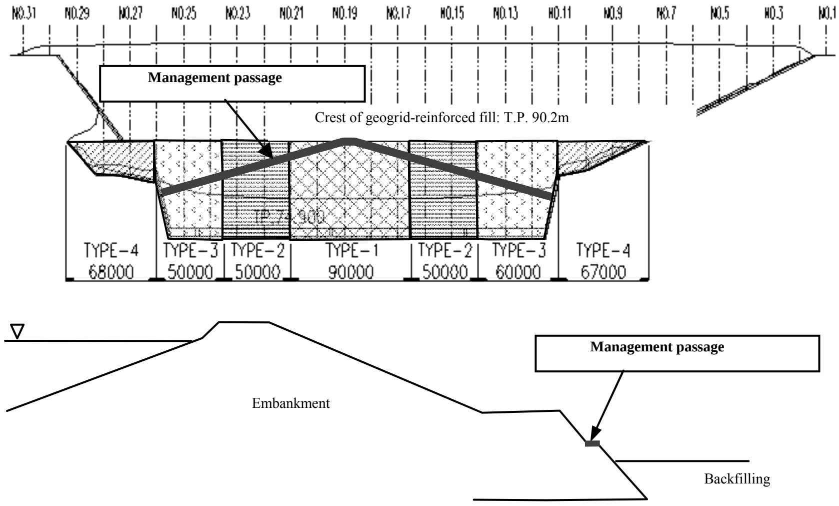


Fig.7. Four types of geogrids used

Photo.4. Primary Reinforcement

Photo.5. Primary Reinforcement

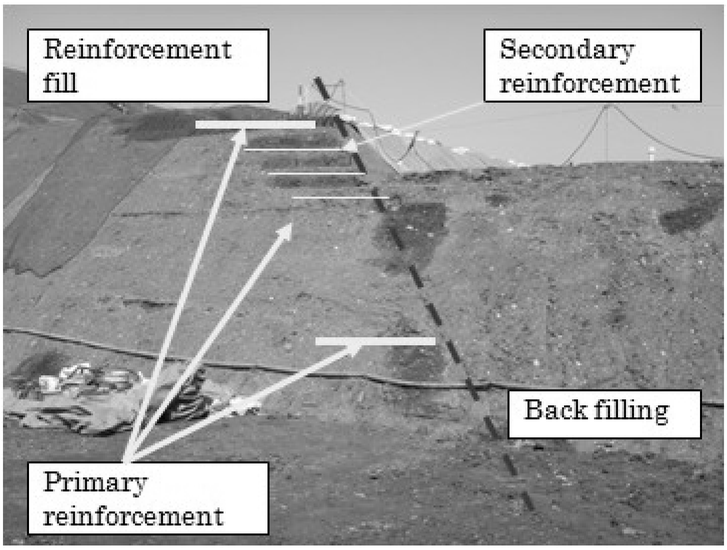


Photo.6. Geogrid Reinforced Steep Slope

Photo.7. Overview of Down Stream Site

4.2 OTHER DETAILS

The lower part for a height of about 6 m of the 17 m-high geogrid-reinforced steep slope (Fig.6) will become below the final ground level after backfilling the area in front of the slope to make the ground level the same as the one of the existing park, Sayama Park. The buried part of the slope was reinforced only with one layer of geogrid as shown in Fig. 6 based on stability analysis.

The face of the geogrid-reinforced steep slope will be vegetated so that the complete slope looks a natural one. The greening works aim at also a protection of the slope face against erosion in case of heavy rainfalls while protecting the geogrid layers against the degradation by UV. It is planned to spray seeds of relevant grass types on a 5 cm-thick layer of substrate medium placed in advance on the slope face.

4.3 LONG-TERM MONITORING OF THE BEHAVIOR OF GEOGRID-REINFORCED STEEP SLOPE

To confirm whether the tensile strains that will develop in the geogrid-reinforced steep slope is kept far below the tensile rupture strain, equal to about 10 %, the tensile strains of geogrid will be observed for a long period after the completion of the slope. The deformation of the slope will also be monitored for some long period. This monitoring is necessary also to confirm that the stability design of the fill is reasonably on the safe side.

Photo.8. Air view of Murayama-Shimo Reservoir after reconstruction

5 CONCLUSIONS

To ensure a sufficiently high seismic stability of an existing earth dam for a very important reservoir for water supply to Tokyo with a densely-populated residential area in front, on the downstream slope of the dam, a counter-weight fill with a17 m-high steep slope reinforced with HDPE geogrid layers was constructed. A 1:1 steep slope was adopted to alleviate a space restraint while to ensure a high seismic stability. The total area of the geogrid layers was 28,500 m^2. As this project is the first case of reinforcing an existing earth dam by means of geogrid-reinforcement, the grading characteristics of the backfill was strictly controlled while a high degree of compaction was ensured. The long-term post-construction behavior of the geogrid-reinforced steep slope will be monitored while ensuring a long-term durability of the geogrid in the steep slope.

6 REFERENCES

Murayama-Shimo Reservoir embankment strengthening technological advisory committee, (2002). "Murayama-Shimo Reservoir embankment strengthening technological advisory committee report"

Foundation of Public Works Research Center (2000). "Design and construction manual of reinforcement soil that uses geotextile revised edition"

Strategy of Shihmen Reservoir High Turbidity

Shen-Hsien Chen, Chaur-Gong Jong, and Chen-Che Lin

ABSTRACT

Shihmen Reservoir, which has been operating for more than forty years, is the major water resource in Taoyuan Area where freshwater supply demands a daily need of about 1,050,000 cubic meters. With a huge catchment of 763.4 square kilometers, fragile geological condition, unsound natural environment and a devastated surface condition through the 921 earthquake in Taiwan, currently Shihmen Reservoir is struggling on the threat of surface erosion by overdevelopment and possible landfalls of typhoons. In 2004, Typhoon Aere brought catastrophic rainfall to the catchment of Shihmen Reservoir and dramatically escalated reservoir turbidity. Consequently, the freshwater supply in Taoyuan Area was adversely affected, and countermeasures on the restoration of freshwater supply were desperately in need. The strategies on the restoration of Shihmen Reservoir can be categorized as "imminent plan" and "median and long term plan". Imminent plan includes immediate construction to improve fresh water supply, the establishment of the task force on monitoring the quality and quantity of freshwater supply, and fighting to remove the restriction of freshwater supply in Great Taoyuan Area (which was finally and completely achieved in 2006). The "Project on the Restoration of Shihmen Reservoir and Related Catchment" was launched for fulfilling the above median and long term plan, which includes assurance of reservoir operation, conservation on the catchment in upstream area of Shihmen Reservoir, and Stabilizing freshwater supply to ensure the daily access to freshwater in Taoyuan Area.

Shen-Hsien Chen, Director, Water Resources Agency, Ministry of Economic Affairs. 12F 41-3 Sec.3 Hsin-yi Rd., Taipei, Taiwan 10651, R.O.C.

Chaur-Gong Jong, Chief, Water Resources Management Division, Water Resources Agency, Ministry of Economic Affairs. 4F 501 Sec.2 Liming Rd., Taichung, Taiwan 40873, R.O.C.

Chen-Che Lin, Associate Engineer, Water Resources Management Division, Water Resources Agency, Ministry of Economic Affairs. 4F 501 Sec.2 Liming Rd., Taichung, Taiwan 40873, R.O.C.

PREFACE

The Shihmen reservoir started in May, 1963 to store water, full water level area 8 square kilometers, effective capacity approximately 250,000,000 cubic meters. There are multi-function of irrigation, power generation, water supply, flood control and tourism. until now ,she operates over 46 years, to promoted agriculture of output, to develop the industry and commerce of Taiwan, to enhance the lives of the people standard, to increase the employment opportunity and to reduce the flood and drought disasters .She has the important contributions for Taiwan.

The Shihmen reservoir original plan mainly supplies the agriculture irrigation water primarily, the volume of supply approximately 800,000,000 cubic meters per year in which livelihood of the people water used only the approximately 3,500 ten thousand cubic meters for 340,000 people and 256 spinning and weaving, chemistry and so on other traditional industry water used, because recent years the Tauyuan area of the industry and commerce to develop fast, the livelihood of the people and the process water increase equal to the irrigation supply volume, to 2007should offer the total quantity, increase to the approximately 1,400,000 cubic meter per day to supply 3,200,000 person and the high tech industrial water used, the supply volume already over 1,000,000,000 cubic meter per year.

Because also the Shihmen reservoir catchment area the hillside is high and precipitous, the brook rapids, the geologic structure are complex,lessened ground due to 921 earthquake of 2000, the Alley typhoon of 2004 causing the soil to greatly collapse in the upper catchment area over 854 hectares, estimated that surpassed 27million cubic meter silt to flow in the reservoir to create the serious siltation, besides had the serious influence to the reservoir normal work function and the life, the high turbidity raw water caused the Tauyuan area to water supply shortage for 17 days, serious influence populace life and industrial function. Therefore, the Shihmen reservoir siltation and the derivation raw water of turbidity high question, must as fast as possible solve, decreases the water scarcity to have the impact.

OPERATE BENEFIT AND CATCHMENT AREA PROFILE

A: Operate benefit

The Shihmen reservoir located at Taiwan north area (Fig 1), dam site located at the Tamsui river branch Tahan river middle reaches, reservoir total length 16.5 kilometers, after Alley disaster, the effective water-holding capacity only remains 218 million cubic meters, the major facility include the dam, the spillway, spillway tunnel, the power plant, the afterbay and the afterbay weir, the Shihmen irrigation canal inlet and Tauyuan irrigation canal inlet. its benefit states as follows take 2007 years as the example.

(A) water supply

The Shihmen reservoir (including public and process water) is supplying water for the Ban-Sin and the Taoyuan area, by the Shihmen, the Longtan, the Da-Nan, the Ban-Sin and the ping-Cnen water purification fields. It supplyies amount of water approximately 1,400,000 M^3 and for 3,200,000 populations every day, in addition has the supply industry special water used like Taiwan oil company of refinery and other research water used like Chungshan Institute of Science and Technology water used and so on, supplies the Taoyuan area 22 large-scale industrial districts, the industrial output surpasses 2,20billion Taiwan dollars.

(B) Irrigation

The Shihmen reservoir construction essential target was for irrigation original .The irrigation scope included the Taoyuan and the Shihmen agriculture association, there are approximately 36,000 hectares farmland, the year supply irrigation water consumption for the approximately 402,730,000 M^3, the rough rice output approximately reached 73,500 tons/year.

(C) power generation

The Shihmen power plant installs 2 groups of the generator. In 2007, power rate approximately 214,000,000 kwh, and I-Xing Power plant power rate approximately 169,000,000 kwh, total 383,000,000 kwh, two electricity generation factory electricity generation strength entrusts the Electricity company to plan the dispatch completely for to sell.

(D) flood control

The Shihmen reservoir design the possible maximum flood 14,500 M^3/s, but the corresponding reservoir expels flood water the quantity is 13,800M^3/s, the design quantity reduces the magnanimity to the downstream to be possible to amount to 700 M^3/s. In the recent 5 years (2003 - 2007) the typhoon events average reduced peak of inflow to lead 22%, the flood peak hinder deter delay equally the approximately 3 hours, She had the flood detention to the reservoir downstream to reduce disaster, to avoid the people loss of life and property .

(E) Tourism

There is fine scenery in Shihmen reservoir, and she is leisure amusement and rest sightseeing paradise of the northern Taiwan, in recent years nearby the reservoir like the Long-Chu bay, the China-Window, the Kind lake and so on have formed to go sightseeing the amusement and rest. From 2005 to now, the Shihmen reservoir tourist population surpasses 1,000,000 people.

Figure 1. Location of Shihmen Reservoir t

B: catchment area profile

The Shihmen reservoir catchment area river system take Tahanxi as a mainstream, originates the Pintian mountain and the Dabajian mountain, has the span 94 kilometers from Shihmen, the catchment area amounts to 764.4 square kilometers, the catchment district by the Tai-Gun brook, the Bi-Sh brook, San-Kun brooks, the Shou-yan brook, the Kui-Hwe brook, the San-Ming brook, Nan-Z-Go brook and so on most branch convergence becomes, the branch total reaches 64 (Fig 2). Basin mean annual precipitation approximately 2,476 millimeters, annual mean run-off approximately 1,400,000,000 cubic meters, nearby reservoir submergence area and dam site sandstone and shale for the Miocene epoch end deterioration, submergence area upstream for miocene epoch early time and the oligocene epoch slight deterioration sandstone and raw shale composition.

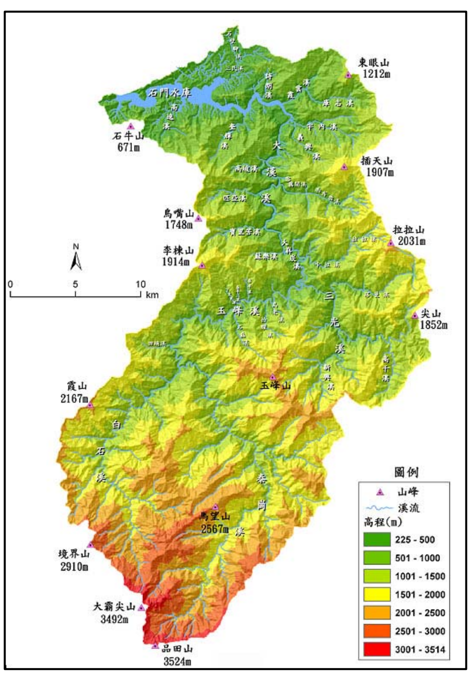


Figure 2. Tributary of Shihmen Reservoir catchment t

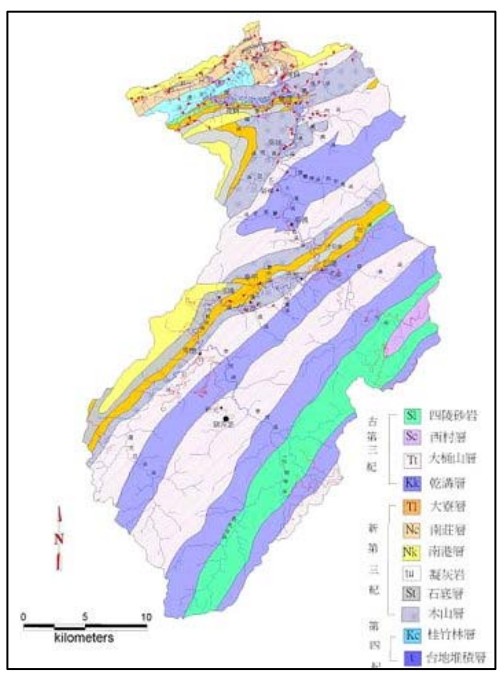


Figure 3. Geologic map of Shihmen Reservoir catchment

THE EFFECTS OF TYPHOONS ON RAW WATER TURBIDITY FROM SHIHMEN RESERVOIR

A: Heavy rainfall caused by typhoons

Typhoon Gloria of 1963, Typhoon Herb of 1996, Typhoon Nari of 2001, Typhoon Aere of 2004 and this year's Typhoon Matsa all have dropped beyond 800 mm of rainfall, causing the soil to greatly collapse in the upper catchment area, and resulting in an abrupt increase in turbidity.

B: Poor geological conditions in catchment area

The Reservoir is situated at the beautiful upper stream where Baishih creek meets Taigang creek, crossed between Siashan and Kuaishan faults. The main formation is the Talu shale of Miocene epoch, Peiliao formation, Shihti formation, Aoti formation and Paling formation of Oligocene epoch. The collapsed area is consisted mostly of Shihti formation and Paling formation near Baishih creek path; and among the two, Paling formation contributes to the most collapsed area.

C: Poor terrain conditions in catchment area

Other than the northwest section of the Reservoir that lies lower and milder, most of the reservoir ground is considered mountain terrain, with elevation from 135 to 3529 meters. Dividing by the level of slopes, sloping grade of less than 30% occupies 10.2% of the reservoir, 30-55% grade occupies 29.5%, and greater than 55% grade occupies about 60.6%.

D: Road construction

Based on the needs for tourism business and land development, the North East-West Cross-Taiwan Highway as well as other county roads have been widen, making the bare land further eroded. The ground sloping along the direction of water slides off easily; improper disposal of waste soil gets washed off by heavy rains.

E: Excessive use of the catchment area

Cutting down the original deep-rooted woods, and replacing the area with shallow planted produce will plant a hidden problem for the future, accelerating soil silting into the reservoir, cutting short the reservoir's lifespan, and increasing the turbidity during typhoon periods.

F: water supply and desiltaion facilities insufficient

Because the Shihmen reservoir early time is by supplies the agricultural water primarily, therefore, its intake design in the lower level, and there is not a desiltation facility, causes the running water purification field handling ability to be insufficient, creates the water scarcity situation.

URGENT STRAIN PROCESSING

After Shihmen reservoir construction, In 1963 GeLeli typhoon's inflow peak was the first, in 2004 Alley typhoon's inflow peak was the second, it's peak discharge amounts to the approximately 8,600 m^3/s, and creates the Taoyuan area water scarcity almost 17 days (on August 27), might count the trade to lose the approximately 110,000,000 Taiwan dollars, the industrial district loses the approximately 4,810,000,000 Taiwan dollars, the sum total loses the approximately 4,920,000,000 Taiwan dollars, lost 273,000,000 Taiwan dollars every day equally. Under the administrative team and the folk public figure cooperate fully, the beginning solves the water scarcity crisis. Picks good strain processing to be as follows:

A: Turbidity monitoring

After the Alley typhoon occurs, the massive rainfalls cause the reservoir the raw water turbidity to increase, to understand in the reservoir region turbidity of change various positions various elevations, namely starts to carry on turbidity of monitor the reservoir, understands course of the typhoon turbidity change course.

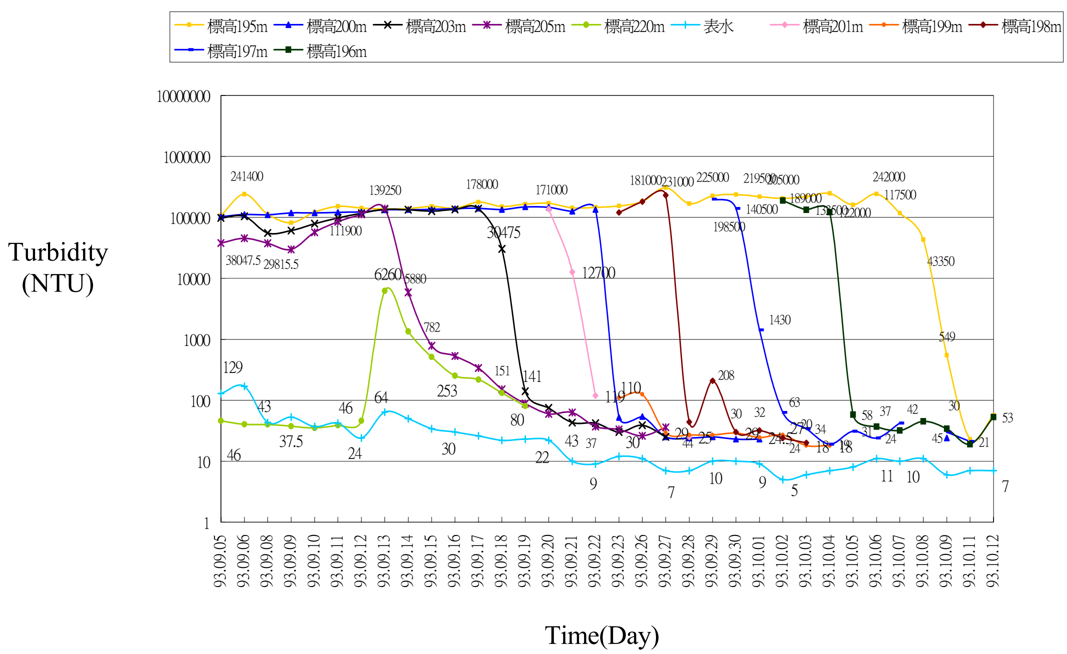


Fig.4 Raw water Turbidity of Shihmen Reservoir on Typhoon Aere

B: Temporary water supply

(A) Set up the user exclusive telephone, for inquiry water supply situation.

(B) To assign nation waterwheel support transports the water, conformity Department of Defense, the Fire agency, Builds agency, the Taipei City government, the Taipei water enterprise place, the Taipei County government, the Taoyuan County government, the state-operated enterprise and the other unit altogether 311 waterwheels, supplies water in the Taoyuan area.

(C) Establishment temporary water distributing point: The water company altogether supposes temporary water distributing point 329; Moreover, Taoyuan County (besides Fuxing Town) 12 villages and towns city geographical unit of government also from supposes for water distributing point altogether 941, equals altogether 1270; The medical service and the livelihood of the people carry water station 3; Process water 12; The fire prevention takes water station 10.

(D) Destacking mineral water: The water company and the Taoyuan County government purchase mineral water 256,044 boxes for the user.

(E) To open water company purification factory for the to carry the water : For in accordance to the taoyuan area industry and the livelihood of the people water used, the water company opens the western area main water purification factory 12 points to provide the water 24 hours of every day.

C: Water Resource assign

(A) The Taipei water enterprise place supports Ban-Sin area (Ban-Sin plan I) the water increases by the approximately 200,000 m3/day to 500,000 m3/day , supports the north Taoyuan area 100,000 tons again by the Ban-Sin water purification factory.

(B) The Da-Nan water purification factory urgent addition handling equipment, water supply ability increases by the original 300,000 m3/day to 390,000 m3/day.

(C) The Afterbay of Shihmen reservoir, the left bank takes the water temporarily project: For in accordance to the south Taoyuan area water supply need, the water company disassembles San-Kan pumping station existing water 3 pumps, Afterbay left bank establishment temporary pumping station. “the Ping-Chen water purification factory second raw water intake plan” not yet completes the water conduit line to lay down temporarily in the road surface. Because although the work site too narrow to construction , in 24 hours of every day under rushing a job, midnight of September 4 finished, may supply water 300,000 m3/day.

(D) The Shihmen reservoir of dam takes the water project: After the Shihmen reservoir the Afterbay left bank takes the water project finished, south Taoyuan area water used temporarily must pick the district water supply, to enable the entire Taoyuan area to resume the normal water supply, handles in the Shihmen reservoir of dam takes the water project, handles urgently by the Water Resource Agency completes erecting of the 24 water pumps, partitions the pump way to take the water supply, lays down the temporary water conduit line in the Shihmen reservoir's campus, links to San-Kan pumping stations, completes on September 9, supplies water approximately 300,000 m3/day.

D: Reservoir operation change

In order to reduce the muddy water to flow nearby the Shihmen reservoir dam, in typhoon period, to operate the reservoir of power generation, permanent river outlet, expels flood water facilities and so on tunnel and dam spillway, the coordinate density current situation, take lower level outlet first opening as the principle, the use water power row of silt way the silt conduction current emissions, avoids the lower level outlet emissions insufficiency, causes the muddy water ahead of time to raise rises.

After 2004, 2005 and so on typhoon events, except schedule “the Shihmen reservoir water supply urgent strain group rules”, the Water Resource Agency and the Taiwan water company and examine the strain mechanism, Before Typhoon events approach are the being established strain group handle fit out and the strain work, and after all previous typhoon events examines the improvement, enables achieves strengthens the early warning to start, the interface conformity, the condition analysis evaluation, the notification confirmation, warning processing and tracing handles and so on matters concerned. In addition handles the dam takes 960,000 m^3/day; The Shihmen water purification field additionally builds 500,000 m^3/day raw water ponds; Yuan -Santhe weir takes raw water facilities and so on, many projects to achieve one goal in the soft hardware, achieve the goal of not district water supply from 2006 years.(Table 1)

E: Following in accordance to measure

In 2006, starts to handle “the Shihmen reservoir and the catchment area improvement plan”(Table 2-4) 6 years of plan and divide 2 stages carries on, until now already completed the 1st stage majority of work (for example : Urgent water supply works and Reservoir modification in middlestream, Stable water supply facilities and promotion of pipelines in downstream, Conversation and management of watershed in upstream), the short-term goal (2006-2008) had achieved the not district water supply goal ; At present (2009) continually will impel the 1st stage not to complete the work also simultaneously to promote the 2nd stage work, completes the lamination to take the water project and to complete the test and the revolution in not the MoLarke typhoon, and the time can achieve at the end of 2011 improves north Taiwan water resources supply ability and flexible, the breakthrough present situation row (clear) the silt quantity to the annual mean 2,190,000 cubic meters, and may promote in the catchment area the round-off work, then the reduced silt output, establishes disaster of reporting chain the consummation, promotes the water furnishing ability effectively, safeguards the populace water used rights and interests, then holds beautiful mountains and pure water, lengthens of service life the Shihmen reservoir and achieves continues forever to manage goal of the use.

IN ACCORDANCE TO GLOBAL CLIMATE CHANGE

As a result of the greenhouse effect whole world warming, for one hundred years average temperature rises 2 ℃, the sea level average rise amounts to 20~40 centimeters, and causes rainfall of state Taiwan to come under the influence, if the Taipei weather station, by nearly hundred annual rainfall material statistics knowledge, the year always rains the date number to reduce year by year, approximately few 27.8 days, but the year total rainfall amount assumes increases the tendency, approximately increases 268 millimeters. In Taiwan area present ten years great drought, under a five year partial drought's situation, will be unable to say for certain the dry time to grow in the future, the arid number of times will increase, speaking of the water resources utilization will produce is not easy to affect. For in accordance to future global climate change influence, so-called “the human will not have the foresight, must have the sorrow near at hand”, development, the dispatch regarding the multiplex water resources, should consider the drought and flood risk, a finer operation, the more effective use water resources, by in accordance to may not anticipate the risk in addition.

Table 1 Raw water Turbidity and Water supply situation of Shihmen Reservoir on Typhoon events

Year	Typhoon name	Peak flow (cms)	turbidity (NTU)	Amount of preciation (mm)	Water supply situation	operation of 9.6*105M3pumping station (hr)
1996	HERB	6,363	—	790.3	Shortage 9 days	
2004	AERE	8,594	208,930	1,042.0	Southern taoyuan shortage 17 days	
2005	HAITANG	3,199	27,800	510.8	Southern taoyuan shortage 1 day	Pumping 4.0*105M3/day
2005	MATSA	5,166	96,400	846.9	Southern taoyuan shortage	Pumping 4.0*105M3/day
2005	TALIM	3,689	46,300	387.6	Supplies water normally	Pumping 4.0*105M3/day
2006	Heavy rain	818	4,935	—	Supplies water normally	16
2007	SEPAT	1,844	5,820	356.2	Supplies water normally	26
2007	WIPHA	2,788	21,159	437.3	Supplies water normally	80
2007	KROSA	5,300	27,930	670.7	Supplies water normally	96
2008	FUNG-WONG	2,039	10,280	273.9	Supplies water normally	29
2008	SINLAKU	3,447	9,500	965.2	Supplies water normally	124
2008	JANGMI	3,292	8,820	427.2	Supplies water normally	67
2009	MORAKOT	1,837	8,112	486.4	Supplies water normally	New intake facility operated 75hr

Table2 Rehabilitation plans of the Shihmen reservoir and its watershed- Projects of Urgent water supply work facilities and rehabilitation of the reservoir

Project item	Achievements
(1)Upgrade urgent pump water ability to 960,000 tons and pipe lines improvement	Finished
(2)Repair power plant and PRO emergency works	Power plant was repaired
(3) Groundwater supporting for water supply in the Taoyuan and Hsinchu industrial park	Assessment of water quality pumpage is not qualified .
(4)Emergency response for low water level supply	Finished
(5)Building new reservoir intake works	Test run of the Project is completed.
(6)Afterbay improvement, prepared ponds and artificial lakes	Project of planning and designing is completed, Engineering will be executing.
(7) Desiltaion facilities Improvement work	Desiltaion facilities at the PRO is finished.In Execution
(8)Extension sediment prevention work	In Execution
(9)Investigation, planning, trial and research.	In Execution
(10) Project of The hydrology and the water quality experiment monitoring center entire constructs	In Execution
(11)Facilities repaired and environment improvement	1st stage projects is completed,In Execution.
(12)Reservoir sediment dredging	In Execution

Table3 Rehabilitation plans of the Shihmen reservoir and its watershed- Projects of Stabilization water supply facilities and improvement of pipelines

Project item	Achievements
(1)Improvement of JianShan pumping station	Finished
(2)Upgrading Shihmen water treatment plant capacity to 500,000 M3	Finished
(3)Extension of Longtan water treatment plant	In Execution
(4) Transporting the water in the Dahan river to the Taoyuan area	In Execution
(5)Mutual water supply between Taoyuan and Hisnchu	Finished

Table4 Rehabilitation plans of the Shihmen reservoir and its watershed- Projects of Conversation and management of watershed

Project item	Achievements
(1)Land use management	In Execution
(2)Land use, environmental ecology, and disaster prevention and monitoring	In Execution
(3) Conservation of the watershed	In Execution
(4) Education for conservation and disaster prevention	Regular work,In Execution

CONCLUSION

A: The Shihmen reservoir catchment area amounts to 763.4 square kilometers broadly, this reservoir present is new for the Taipei and Taoyuan most important water supply originates, and does not have other water sources to be possible to substitute, will have the question to create serious water supply being out of balance slightly, therefore the upstream - reservoir catchment area the management, the middle reaches - reservoir function improvement and the operation optimization, the downstream - water purification and pipe network dispatch ability, needs to coordinate mutually, will be linked together indispensably, various units must the intercoordination cooperation, achieve the early warning to start, the interface conformity, the condition analysis evaluation, the notification confirmation, policy-making processing truly to be clear about and tracing handle the situation, will transmit, the effective conformity mutually all quarters information, to reduces the water scarcity risk.

B: At present "the Shihmen reservoir and the catchment area improvement plan" has aimed at faces the catchment area earth stone avalanche, the storehouse district silt siltation, the reservoir water supply to be unbalanced and the water conservancy facilities damage and so on four difficult problems positive handle promptly, the 1st stage achieve not district water supply goal; After completing, achieves the reservoir catchment area care, promotion stable water furnishing ability and lengthens the reservoir life, guaranteed that the populace live the blessing.

C: Will not have the water not to have the future, the water resources is global each person must together question of the attention. Therefore, water resources effective management, is the balanced supply and demand, the development of reservoir or the pond for the inevitable behavior.

Is rich for Taiwan various basins catchment area silt output, constructs the reservoir in the river main channel, because the silt siltation is serious, lessens the reservoir life, also has the influence regarding the downstream river and the coast sand source supplies, has the negative impact on the environment. Therefore, implementation and the earth granulated substance integrated management countermeasure the reservoir against silt measure, coordinates the multiplex water resources management (IWRM) to reduce in addition to the reservoir relies on, use seawater desalination, ground and ground water union utilization and so on multibarreled uneven, examines decides the water resources question, the profit is obviously important.

D: Existing reservoir function enhancement and the maintenance for the next water resources management key emphasis in work, after especially will contain water resources development costs and so on newly built the reservoir to elevate successively, will impel not easily. The reservoir continues forever to manage appears day by day important, especially by silt processing, maintenance and the security check the dam and the related structure equipment, protection of the catchment area and the related of environment and so on a work is most important, but the new water source's development should take seriously similarly, or should first provide for a rainy day the plan to handle.

REFERENCES

[1] 台灣省石門水庫管理局，1968，石門水庫集水區治理調查規劃報告。
[2] 台灣省石門水庫管理局委託辦理，1990，石門水庫集水區第二階段治理規劃。
[3] 經濟部水利署北區水資源局，2002，「經濟部石門水庫運用要點」。
[4] 賴伯勳、鍾朝恭、王瑋，2005，颱風期間石門水庫濁度處理與應變措施，水利期刊，第15期。
[5] 經濟部水利署北區水資源局，2006，「石門水庫營運四十年成效分析」。
[6] 鍾朝恭、郭耀程、林震哲、高憲彰，2006，石門水庫緊急抽水設施對壩體之影響與分析，水利期刊，第16期。
[7] 巨廷工程顧問股份有限公司，2006，「石門水庫既有設施防淤功能改善工程計畫」，經濟部水利處北區水資源局。
[8] 行政院，2006，石門水庫及其集水區整治特別條例。
[9] 行政院，2006，石門水庫及其集水區整治計畫第 1 階段執行計畫（核定本）。
[10] 賴伯勳、鍾朝恭、郭耀程、林震哲、許勝田，2007，石門水庫增設取水工之設計理念，水利期刊，第17期。
[11] 林震哲、郭耀程，2007，石門水庫漂流木問題與攔污索設置，水利期刊，第17期。
[12] 行政院，2008，石門水庫及其集水區整治計畫第 1 階段修正執行計畫（核定本）。
[13] 行政院，2008，石門水庫及其集水區整治計畫第 2 階段執行計畫（核定本）。

Earthquake-resistant Distribution Pipeline Project in Nagoya City

Toshichika MATSUI

ABSTRACT

In 2002, Nagoya was designated as an area subject to strengthened earthquake disaster prevention measures as a result of review of seismic center areas for the anticipated Tokai Earthquake. Accordingly, districts in which earthquake-resistant pipes are to be used have been expanded from the southwest soft-ground regions of the city to its entire service area, as enhancement of the existing seismic measures. However, making huge lengths of water pipelines all earthquake-resistant requires enormous amounts of funds and labor.

The Bureau is taking efficient steps to make distribution pipelines earthquake-resistant, ensuring equilibrium in achieving two goals: provision of first-aid water supply in the event of an earthquake and seismic damage minimization.

The Bureau aims to ensure post-earthquake first-aid water supply by selecting first-priority pipelines and making them earthquake-resistant through intensive work. In other words, we aim to implement seismic measures linearly on distribution pipelines.

Furthermore, we conducted a quantitative assessment of pipelines to take seismic measures systematically in accord with the anti-degradation measures for main distribution pipelines. Assessment results are used as indicators for selecting engineering methods. An attempt is being made to speed the improvement process and to level the project cost through renewal of important pipelines by replacement work, while extending the service life of less important pipelines through "refresh work".

Toshichika Matsui, Distribution Division, Pipeline Department, Technical Headquarters, Nagoya City Waterworks and Sewerage Bureau, 3-1-1, Sannomaru, Naka-ku, Nagoya City, Aichi 460-8508, Japan

OVERVIEW OF WATER SUPPLY IN NAGOYA

The water supply in Nagoya, sourced from the clean and abundant flow of the Kiso River, entered service in 1914 from the Nabeyaueno purification plant. In later years, the waterworks has undergone eight expansion projects, keeping pace with the city's urban development. The served population has reached more than 2,370,000.

Water is taken from the Kiso River at two points: the Inuyama intake point in Inuyama and the Asahi intake point in Ichinomiya. The Inuyama intake point sends water to the Kasugai and Nabeyaueno purification plants. From the Asahi intake point, water is sent to the Oharu purification plant.

Purified water is distributed directly or via eight distribution plants and three pump stations in Nagoya. Figure 1 shows a system diagram of the water supply of Nagoya.

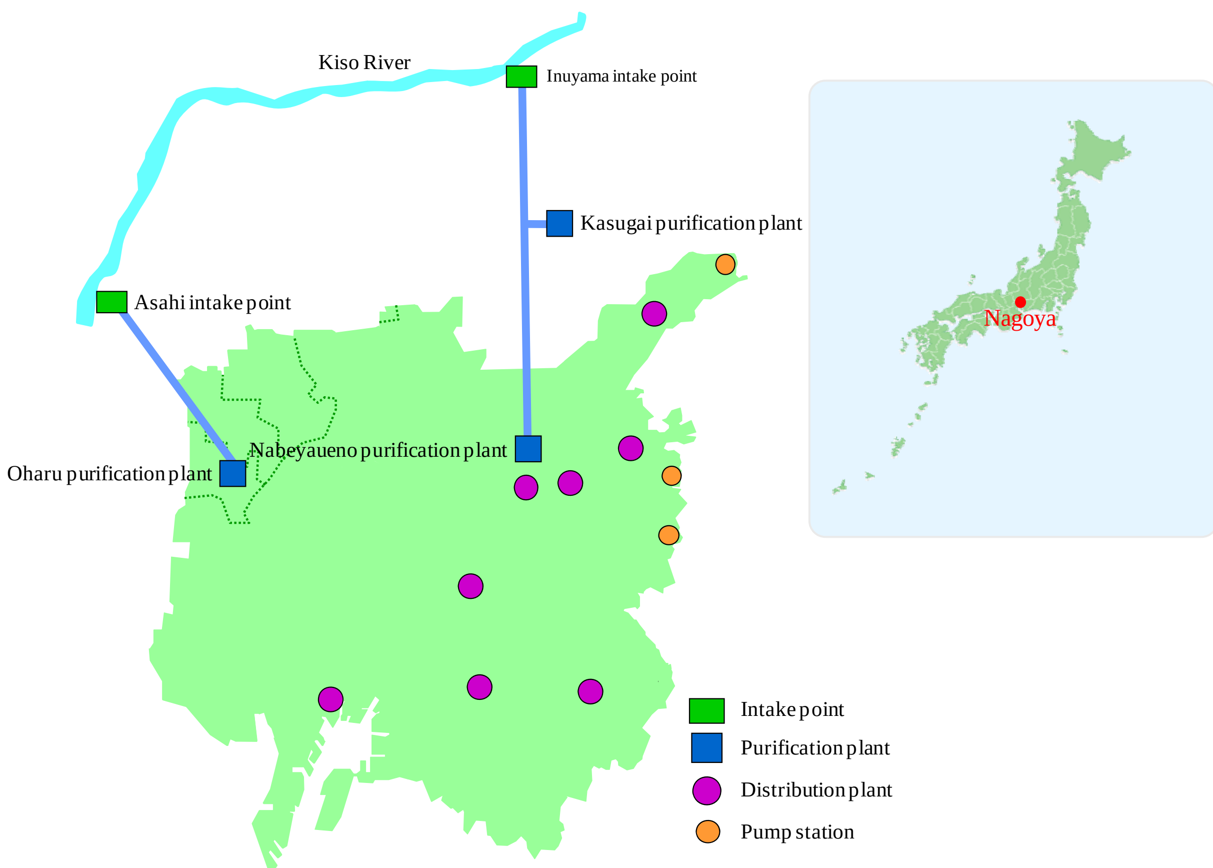


Figure 1. Waterworks system

Topographically and geologically, Nagoya has gently rolling hills in its eastern region, where the ground is relatively stable, and flat lands in the western region where the ground is soft.

In the eastern region, where ground level differences prevail, distribution districts are set up according to ground levels to minimize differences in distribution pressure. Water is supplied by distribution plants located on a one–plant–one-district basis. In the western region, which is virtually free from contoured areas, relatively large distribution districts are set up.

CURRENT STATUS OF DISTRIBUTION PIPELINE

Distribution Pipe Installation Situation

The total length of distribution pipes in Nagoya is presently more than 8,000 km. Following the rise of concern in 1975 about a Tokai earthquake, we at Nagoya began to use earthquake-resistant pipes in 1977. In 1981, the southwest soft-ground areas were designated as seismic reinforcement areas (accounting for approximately 20% of service areas), in which we have installed earthquake-resistant pipes.

In 2002, Nagoya was designated as an area subject to strengthened earthquake disaster prevention measures as a result of review of seismic center areas for the anticipated Tokai Earthquake. Accordingly, districts in which earthquake-resistant pipes are to be used have been expanded to the city's entire service area as an additional reinforcement of seismic measures. As a consequence, the ratio of earthquake-resistant pipelines, as defined in the guidelines for water services set forth by the Japan Water Works Association, has reached 28%. However, enormous amounts of funds and labor are required in order to make the rest of the pipelines (approximately 70%) earthquake-resistant.

Anticipated Earthquakes and Forecast Damage

Table 1 shows anticipated earthquakes and earthquake intensities in Nagoya. It is highly probable that these earthquakes will occur in the near future. Taking seismic measures is of pressing urgency in Nagoya.

Table 1. ANTICIPATED EARTHQUAKES IN NAGOYA

	Tokai earthquake	Tonankai earthquake	Tokai-Tonankai combined earthquake	Nobi earthquake
Intensity scale	4 to 6 lower	5 lower to 6 upper	5 lower to 6 upper	5 upper to 7
Peak ground acceleration (gal)	329	542	542	880

Table 2 summarizes distribution pipe damage forecasts for major earthquakes. The challenge in promoting seismic measures on distribution pipelines in the future is how to reduce the amount of damage and the number of residences affected by water outage.

Table 2. DISTRIBUTION PIPE DAMAGE FORECASTS

		Tokai earthquake	Tonankai earthquake	Tokai-Tonankai combined earthquake	Nobi earthquake
Size of distribution pipe damage (number of locations)		510	1,040	1,220	1,980
Number of residences affected by water outage	Immediately after earthquake	73,200	209,300	255,200	457,800
	Four days later	33,900	125,100	159,500	210,400

EARTHQUAKE-RESISTANT DISTRIBUTION PIPELINE PROJECT

To make distribution pipelines earthquake-resistant efficiently, providing equilibrium is needed in achieving two goals: provision of first-aid water supply in the event of earthquake and minimization of earthquake damage. Approaches taken by Nagoya City to make distribution pipelines earthquake-resistant are reported below.

Selection of High-priority Candidate Earthquake-resistant Pipelines

It is highly likely that Nagoya will experience a major earthquake in the near future. This leads to the need for early incorporation of anti-seismic measures with maximum effects, because making all pipelines earthquake-resistant requires huge amounts of funds and labor.

Making First-aid Water Supply Pipelines Earthquake-resistant

If an earthquake occurs, water is transported and supplied by vehicles. However, it is expected that equal water supply services will not be available due to traffic hindrances or for other reasons and the volume of water will not meet public demand until pipelines are restored. A solution to this problem could be multi-centered first-aid water supply. Using designated large evacuation sites and evacuation centers, 202 first-aid water supply facilities have been developed to serve all citizens with water equally within a distance of approximately 1 km from their respective homes. Each first-aid water supply facility has an underground fire hydrant. In the event of an earthquake, the Bureau staff will install temporary hydrants to serve as first-aid water supply points. Moreover, at elementary schools that serve as post-earthquake evacuation centers and local disaster-relief stations, 265 provision underground hydrants equipped with emergency water taps have been installed for citizens to operate and get drinking water. Pipelines leading to these first-aid water supply facilities and to elementary schools were selected as first-aid water supply pipelines, which have been given priority over other pipelines for taking seismic measures.

Furthermore, pipelines leading to important facilities such as ward offices and other public offices that will function as stations for water transport and supply and for first-aid restoration, also have been made earthquake-resistant.

Our next goal is to ensure provision of water in the event of an earthquake to facilities used by “those who need assistance during a disaster.” More specifically, we will take additional seismic measures for pipelines that lead to medical institutions such as emergency hospitals and dialysis clinics and social welfare facilities.

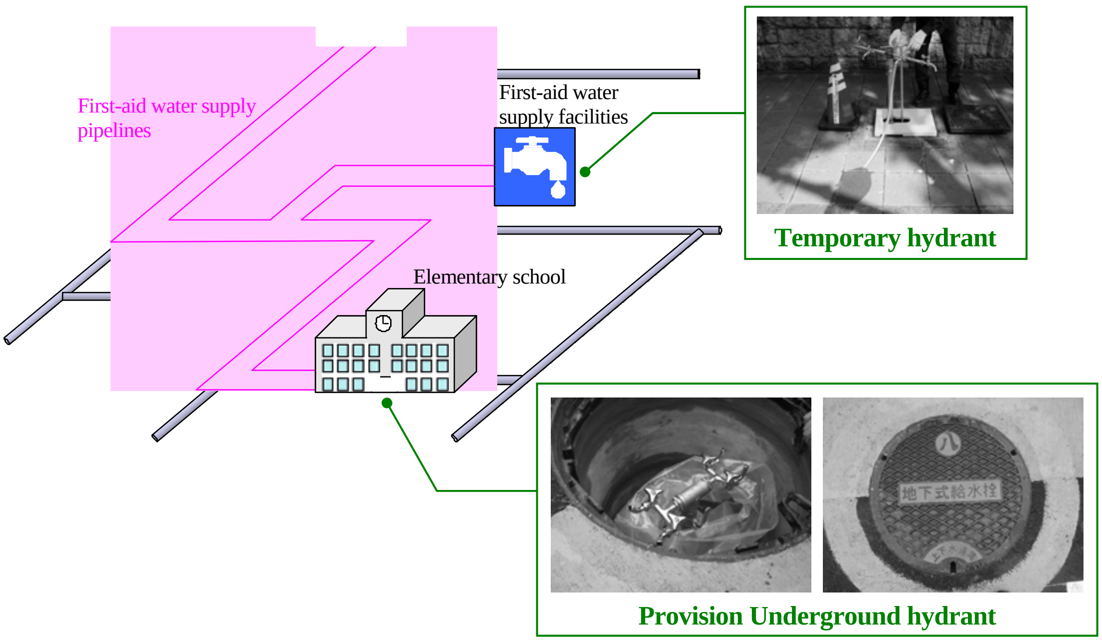


Figure 2. Earthquake-resistant first-aid water supply pipelines

Target Management

Since around the first use of earthquake-resistant pipes, seismic measures had been implemented linearly on the distribution pipelines. However, the progress was not so favorable. To address that situation, our present goal in making the first-aid water supply pipelines earthquake-resistant is to complete the development by 2010 so as to achieve the goal early to ensure provision of first-aid water supply in the event of an earthquake. Moreover, milestone targets have been set for each fiscal year, which have been announced to the general public. The result is successful implementation of seismic measures on 91% of the first-aid water supply pipelines, as shown in Table 3.The step-wise implementation of the project with concrete development targets, as mentioned above, is a sure way to achieve the ultimate goal, because through such an approach our seismic measures become easy for citizens to understand and the staff have a higher motivation.

Table 3. PROGRESS IN MAKING FIRST-AID WATER SUPPLY PIPELINES EARTHQUAKE-RESISTANT

	Number of facilities (number of sites)	Total length of target pipelines (km)	Total length of earthquake-resistant pipes (km)	Ratio of earthquake-resistant pipelines (%)
First-aid water supply facilities	202	330	300	91
Elementary school	265			
Hospital serving as medical station during disaster	14			
Ward office/other public office	36			

Quantitative Evaluation of Pipelines

In Nagoya, distribution pipes of 450 mm or more in inside diameter are ranked as main distribution pipelines, whose installation length reaches approximately 380 km. The impact of damage on the main distribution pipelines caused by earthquake is immeasurable. It is necessary to make the main distribution pipelines earthquake-resistant early in order to minimize earthquake damage. Of the approximately 380 km of main distribution pipelines, 80 km are rated as noticeably aging. Seismic measures are taken on these pipelines in parallel with the anti-deterioration measures on main distribution pipelines on the basis of quantitative evaluation made in a numerical system of rating in two categories: general physical evaluation and importance evaluation.

Background to the Quantitative Evaluation

The total installation length of main distribution pipelines increased quickly in the 1960s and 1970s through rapid urbanization as a result of the sharp economic growth. Almost all standard and high-grade low-strength cast iron pipes have been replaced, while some ductile cast iron pipes with brittle joints remain. Concerns are rising about potential earthquake damage to these pipes. These pipelines are in excess of 80 km in total length. A major challenge for us is how to implement anti-deterioration and seismic measures. Some indicators are needed for formulating a development plan.

Assessment Method

Ductile cast iron pipes with socket-and-spigot joints were selected for evaluation. Nodes were set up at main distribution pipeline division points and service district boundaries to establish evaluation sections between nodes, as shown in Figure 3. The reasons for this are that the pipeline flow rate and other factors change at each division point resulting in different evaluation results and that, at the time of installation, division points often serve as work section boundaries.

The assessment comprised two categories: general physical evaluation and importance evaluation. Factors shown Table 4 were used as evaluation indicators. In the general physical evaluation, pipelines were rated in terms of physical properties such as years in use and burial environment. In addition, the evaluation indicators included possible damage and danger in the event of Tokai, Tonankai and Nobi earthquakes. These are evaluation indicators needed for developing seismic measures. For the importance evaluation, pipeline network analysis was used. Pipelines were evaluated in terms of relative importance with regard to the maximum flow rate and backup availability of the target section. Also rated was the degree of contribution to water supply, or how much the target section distributes water to first-aid water supply facilities and other facilities that require water supply service in the event of an earthquake.

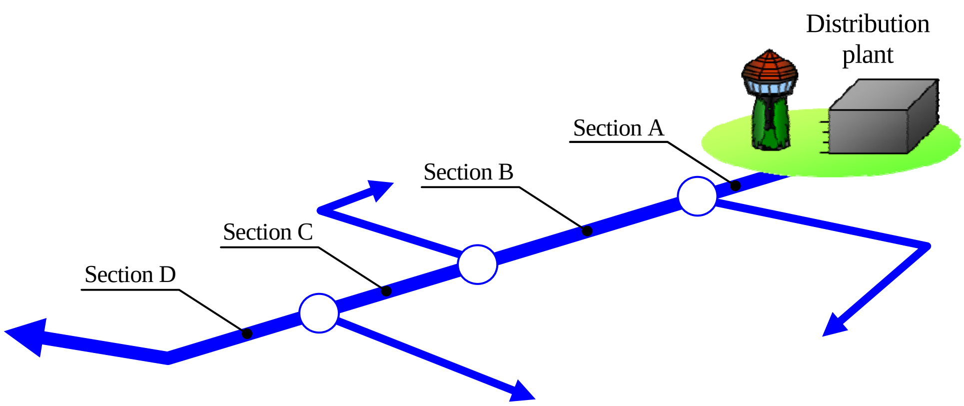


Figure 3. Pipeline sectioning for evaluation

Table 4. EVALUTION INDICATORS

General physical evaluation indicators		
Rating by physical property	· Years in use · Pipe material · Joint type	· Lining · Exterior corrosion-proof measures · Burial environment
Rating in terms of seismic measures	· Relative danger after Tokai earthquake · Relative danger after Tonankai earthquake	· Relative danger after Nobi earthquake · Road class
Empirical rating	· Past leakage and rupture history	
Importance evaluation indicators		
Ordinary situation	· Maximum flow velocity · Maximum flow rate	· Degree of water supply contribution to large-volume customers · Backup availability
Post-earthquake situation	· Degree of water supply contribution to first-aid water supply facilities	

Selecting Suitable Engineering Methods

Figure 4 shows a rendered image of pipelines classified according to the quantitative assessment. The general physical evaluation reveals the degree of aging and post-earthquake danger. The results are used as a yardstick as to whether an early response is needed or not. The importance evaluation results present a yardstick for determining whether, for improvements, costly renewal or inexpensive life-extending measures should be implemented. Based on the results, an engineering method is selected for achieving improvements.

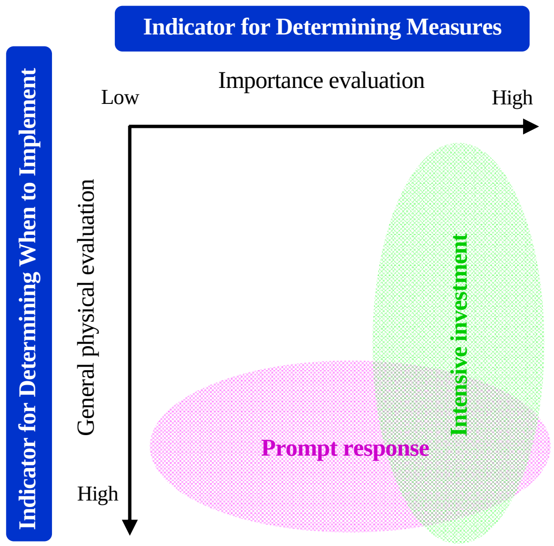


Figure 4. Rendered image of pipeline classification through quantitative evaluation

Engineering Method Selection Procedure Using Assessment Results

Ductile cast iron pipes with socket-and-spigot joints accounted for more than 20% of main distribution pipelines. Since the rate of improving main distribution pipelines has recently remained at 3 km per year, it was necessary to increase the rate. However, due to increasing urbanization and congested underground facilities making excavation difficult, and tight financial conditions resulting from decreases in water usage, it was found difficult to speed the improvement if replacement work and other renewal measures only were used. Therefore, renewal measures were combined with life-extending refresh work measures to speed the improvement process and level the project cost.

Figure 5 shows an image rendering of engineering method selection using quantitative pipeline assessment results. The importance rating “1” denotes highly important pipelines, which are to be renewed through replacement work. Rating “3” represents pipelines of low importance, which are to undergo refresh work. Thus the indicators are used to select suitable engineering methods.

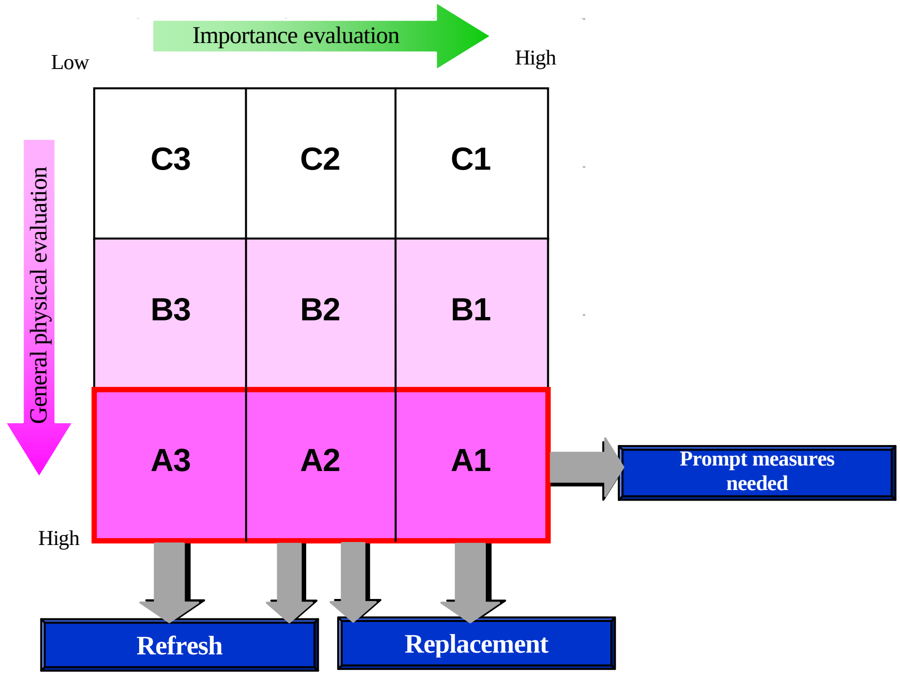


Figure 5. Image rendering of engineering method selection

Refresh Work Overview

Aside from the conventional method of renewal by reinstallation, the lining hose method and internal reinforcement method for joint leakage prevention and earthquake resistance improvements are categorized as "refresh work." Life-extending measures incorporating refresh work are used to enhance anti-deterioration and seismic measures.

Refresh work is intended to extend the service life of existing ductile cast iron pipes. Compared with reinstallation and other renewal measures, refresh work is less costly and can be easily implemented. Refresh work includes two types.

One is the lining hose method, which had already been proven on branch distribution pipes in Nagoya. In this method, pits are dug at both ends of a work section. The lining material, a fiber reinforcement coated with polyethylene resin, is inverted by compressed air and affixed on the internal surfaces of existing pipes. This method prevents joint leakage and rust in water caused by deterioration of inner pipe surfaces. Furthermore, in the event of joint dislocation caused by earthquake, internally sealed pipes will prevent leakage, ensuring improved earthquake resistance.

The other method is known as the internal reinforcement method. Conventionally, a rubber band was installed on the inner surface at joints for leakage prevention. The internal reinforcement method is an innovative method developed on the basis of the conventional method. It is the addition of joint displacement resistance to the conventional method. More specifically, holes approximately 20 mm in diameter are bored toward the joint socket, stainless steel pins are inserted, and a rubber band is installed on the pipe interior. The shearing strength of the pins prevents the spigot from coming off the socket. The force needed to remove the spigot reaches more than 1.5DkN (where D is the diameter in mm of the pipe being worked on), which is half that of an earthquake-resistant joint. Since the work requires workers to enter the pipeline, this method is implemented on pipelines approximately 900 mm or more in inside diameter. Figure 6 shows a schematic diagram of the internal reinforcement method.

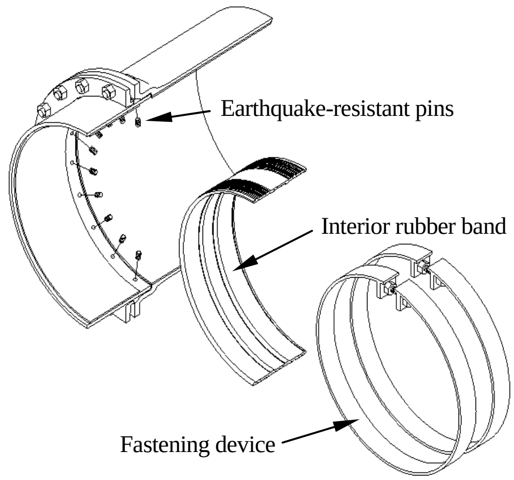


Figure 6. Schematic diagram of internal reinforcement method

Effects of Refresh Work

Refresh work is, nonetheless, a life-extending measure. It is inferior to reinstallation in terms of earthquake resistance and durability. However, refresh work supplementing replacement work is expected to result in the following effects.

- Involving little need for excavation, the method is less costly than replacement work and causes only limited impact on traffic during work. This leads to an increased pace of improvement, enabling early implementation of measures.
- The 1960s were years of expansion, in which pipelines were installed intensively. At present, they are undergoing an aging problem. Even if we manage to renew these pipelines in a short period of time, the same challenge will recur in several decades. Refresh work enables decentralized renewal timing and leveled project workload, avoiding large-volume renewal cycles.

CONCLUSION

Water usage growth has been stagnant due to the spread of water-saving awareness in society and popularization of bottled water. Accordingly, water utilities have been facing financial difficulties. However, pipelines are aging each year and the aging problem needs to be addressed. Despite the tight financial conditions, the municipal government of Nagoya has used earthquake-resistant pipes and implemented seismic measures since an early stage. Consequently, the ratio of earthquake-resistant pipelines, as defined in the water utility guidelines established by the Japan Water Works Association, has reached 28%. The ratio has remained at a relatively high position in Japan. We proudly believe that the seismic measures taken in Nagoya represent an advanced approach. The Bureau intends to promote seismic measures on distribution pipelines through the project reported in this paper. We will build earthquake-resistant distribution pipelines from a broad perspective, with foresight and without delaying the problem.

Memphis Light, Gas and Water Seismic Planning: The Role of Tanks, Generators, and Distribution System Piping

Chandrika Winston [1], Fred Von Hofe [2] and Quinton Clark [3]

ABSTRACT

In 2006 MLGW hired R. W. Howe & Associates, a seismic risk consultant, to perform a Multi-hazard Risk Assessment, including seismic risk, for MLGW's water, electric, and gas systems. The risk assessment focused on a system-wide approach as opposed to being limited to a specific component vulnerability assessment with the focus being on system vulnerabilities potentially resulting in significant disruptions to MLGW's operations and loss of service to customers.

As part of this risk assessment, the consultant was asked to investigate several levels of seismic event scenarios focused in the New Madrid Seismic Fault Zone. The consultant performed a comprehensive study of our water distribution system. It was noted that Memphis has a relatively unique system, in that our water is obtained from an artesian source (rather than impoundment or collection of surface waters). Water is pumped from wells through eight major water pumping stations/treatment facilities and hence through a highly interconnected distribution system to deliver the water to our customers.

This presentation focuses on several relatively unique but important aspects of MLGW's water system and related seismic performance objectives:

- Emergency generators for high service pumps
- Water distribution system considerations
- Plastic storage tanks used for storage of chemicals used in the water treatment process

The presentation will focus on considerations relative to seismic risk mitigation for the above MLGW water system components.

CHEMICAL STORAGE

Chemical storage is a very important aspect of seismic planning. In the case of a major seismic event sodium hypochlorite, the chemical used for disinfection of the water, would play a major role in protecting the water from microbiological contamination and ensuring MLGW's customers the water was safe to drink. There are a number of considerations with protecting a chemical tank from failure (tank toppling or rupture of tank and/or connections and leakage) in a major seismic event. Considerations include tank construction material (plastic or steel), seismic restraint system (whether manufacturer's proprietary system or engineered for retrofit), piping connections, tank foundation, and containment scheme.

MLGW investigated steel versus plastic tanks. It was determined the plastic tank was the most economical solution since MLGW, unlike many other utilities, has 8 major and 3 minor water treatment plants. Chemical storage tanks are made from several materials: fiber reinforced

[1] Manager, Water Matrix, 220 South Main, Memphis, Tennessee 38103, cwinston@mlgw.org

[2] Acting Manager, Water Quality Lab, 220 South Main, Memphis, Tennessee 38103.

[3] Supervisor, Water Engineering and Operations, 220 South Main, Memphis, Tennessee 38103.

polyethylene, high density polyethylene, and high density cross linked polyethylene are a few of the options available. MLGW selected high density cross linked polyethylene for its strength and resistance to sodium hypochlorite. In a major seismic event, the high density cross linked polyethylene offered the best chance for surviving the impact force from sloshing inside the tank and the force from the manufacturer's cable restraint system which includes large stainless steel restraint clips at the base of the tank.

There are a number of chemical storage tank designs available from various manufacturers. Initially, MLGW selected a Poly Processing Design called a Safe Tank. Basically, this is a tank with in a tank. If the inner tank wall were to rupture or fail, there would be a second wall to contain the chemical. This is an excellent tank and offers a great deal of containment security but is very difficult to clean. With respect to sodium hypochlorite, cleaning the tank on a regular schedule is important because the chemical undergoes decomposition when exposed to sunlight and temperature. The decision was made when MLGW decided to convert from chlorine gas to sodium hypochlorite to locate all tanks outside. Seismic considerations played a role in this decision because most of MLGW's water treatment plants are of older construction and would be difficult to retrofit for large chemical storage tanks.

Because of the cleaning issue with respect to the sodium hypochlorite chemical storage tanks, MLGW make the decision to look for an equally strong chemical storage tank that was easier to clean. Poly Processing had just the tank design recommended for sodium hypochlorite and addressed the cleaning issue. The Integrally Molded Flanged Outlet (IMFO) tank allowed easy cleaning. Figure 1 shows a picture of the flange outlet portion of the IMFO tank. The tank is, as seen in the picture, mounted on a pad. Note that all that is needed to clean the tank is flushing it. It is not necessary for an employee to enter the tank for cleaning purpose. The IMFO tank also addresses another issue that was a minor problem. The transition fitting is not needed with this type of tank because of the flanged outlet. Note also that the expansion joint also attaches to the transition fitting.

As shown in the figure all of MLGW's sodium hypochlorite tanks are located on reinforced concrete foundations. The tanks are also surrounded by a reinforced containment wall in case there is a major event like a spill. The reinforced concrete foundation also serves to provide anchorage for anchors called for by the manufacturer's seismic restraint system and contributes to overturning resistance by virtue of its mass.

Figure 1 Integrally Molded Flange Outlet Tank on Pad

Another important consideration is that the tank must be easy to remove and replace when required by age or other considerations.. Both the Safe Tank and the IMFO tank are easy to remove as long as the manufacturer's seismic restraint system is installed. Initially, MLGW used only a seismic restraint system recommended by the manufacturer.

For a number of years MLGW has been in the process of seismically retrofitting its major water treatment plants. A seismic risk consultant was hired to design the retrofit of these plants which included the bulk storage tanks. The consultant recommended a steel post-and-brace-type seismic restraint system with steel bands surrounding the tank for both the sodium hypochlorite and the hydrofluorosilicic bulk storage tanks. Figure 2 shows one of MLGW's bulk storage tanks retrofitted with the band type seismic restraint system.

Figure 2 Safe Tank with Band Type Restraint System

One of the chemical storage tanks retrofitted with the band type seismic restraint system had to be replaced shortly after the retrofits were completed. It was immediately recognized that the band type seismic retrofit was not operation friendly or economically friendly. The band had to be cut and removed to permit tank removal. Additionally, the simple process of removing the tank was now more complex because the tank had to be lifted above the bracing. Along with the construction cost and operational difficulty associated with the band type seismic restraint system, a third issue arose. The manufacturer of the plastic tanks visited our sites and noticed the tanks with a band type seismic restraint system and pointed out that his literature states, "No restraint or lateral attachment bands circumscribing the tank are allowed." The manufacturer explained that in a warm climate such as Memphis the tank can undergo expansions and the band could damage the tank. Secondly in the case of a major seismic event, the support beams and the band could also damage the tank.

In 2007 MLGW hired a nationally known consultant to do a multi hazard risk assessment. The consultant made the recommendation that the band type seismic system not be used with the plastic tanks and that MLGW return to the use of the manufacturer's recommended seismic system of stainless steel restraint clip and anchors.

Subsequent consideration as part of further seismic risk assessment study indicated that there are presently no seismic restraint systems for large flexible (plastic or poly) tanks such as those favored for MLGW uses that offer a high level of assurance of leak-free post-quake performance. This being the case, alternate or additional/back up seismic risk mitigation measures should be considered. Such measures include:

(a) maintaining operating fluid volume/fluid level at less than (seismically) critical level (say, less than 2/3 full). This would require more frequent filling or additional tank(s) utilized in a similar fashion.

(b) maintaining a spare empty tank (ideally, the future replacement tank) close by with fittings in place to accommodate ready transfer of contents in the event that the in-service tank is damaged and leaks (failure is not likely to be wholesale rupture and abrupt loss of contents but more likely leakage)

(c) Lastly, it is recommended that utility seismic performance objectives be carefully assessed and made known to tank suppliers and compared with their seismic restraint design criteria (ideally, IBC latest edition with I=1.50 for components critical to operations).

EMERGENCY GENERATORS

MLGW uses diesel power generators to provide emergency power for its water pumping stations. Presently, MLGW has generators at seven of its eight major pumping stations. One of the first generators was at Shaw Water Pumping Station in 1989. The 1250 kW generator was included in the original design of the station. Subsequently, 1500 kW generators were added at Mallory and Sheahan Water Pumping Stations in 1991. Two more 1500 kW generators were added at Davis and Lichterman Pumping Stations in 1998 and 2000. The last two 2000 kW generators were installed at McCord and Morton Pumping Stations in 2005 and 2007. MLGW is presently installing an additional 2000 kW generator at Shaw Water Pumping Station. MLGW also installed 20,000 gallon underground storage tanks for diesel fuels.

Figure 3 Typical Generator on Site

MLGW determined it was necessary to install the emergency generators to provide back-up power for not only the water pumping stations but also for the wellfields at each station. Since Memphis is in the New Madrid Seismic Fault Zone, the threat of an earthquake would possible impede the ability of MLGW to provide water for domestic consumption and for fire protection. All of the eight major pumping stations are served by at least two electrical feeds from different substations with the exception of Allen and Shaw. Allen is actually served by three feeds from two different substations. Shaw is served by two feeds from the same substation. A normally open tie breaker will allow each station to be fed by one circuit if necessary.

Each pumping station has 20 to 30 wells in each wellfield. As stated earlier, most of the generators have the capability of providing power to select wells at each pumping station. The majority of the wells at Allen are actually on a dedicated underground network fed from two breakers at the pumping station. Shaw actually has five of its wells feed on a dedicated underground network fed from a breaker at the pumping station. Mallory, Davis, Lichterman, McCord, and Morton actually utilize an electrical field switching scheme to feed 5 to 8 wells in each respective wellfield. Providing power for select wells is necessary in order to replenish the 10 MG to 20 MG underground reservoirs at each pumping station.

The pumping stations have a treatment capacity of either 30 or 35 MGD. Each pumping station has 4 to 6 high service pumps. Each high service pump is rated 10, 10.5, 12.5 or 15 MGD. The motors for the pumps range from 400 to 700 horsepower. The emergency generator can either supply power to two high service pumps or one high service pump and up to seven wells. The generators can also be placed online in parallel with the electric utility.

The generators have several seismic features which are required per the specifications. One design requirement is for premium neoprene vibration isolators. The isolators are sized to support the static weight of the engine-generator plus additional torque and vibratory loads imposed by the running unit. The isolators incorporate seismic restraining or dampening devices to limit motion.

The generators have a 660 gallon capacity fuel tank mounted inside the Power Module. The fuel tank is secured to the floor of the power module with hardware designed to withstand lateral forces to meet local seismic requirements. The weight of the fuel tank at full fuel capacity and all accessories are utilized in the calculation of the lateral forces.

The generators systems, peripheral support components, and all foundations are seismically designed in accordance with local seismic requirements. One of the specified requirements is for the emergency generator systems to be "fully operational" following a magnitude 7.0 earthquake. The equipment components included for seismic design are:

- Engine generator set and frame mounted components including the base frame and base vibration isolators
- Engine exhaust system and supporting structure
- Engine start batteries, charger and supporting structure
- Fuel oil day tank and supporting structure
- Engine and generator control panel/switchgear
- Distributed systems including connected electrical conduit, cabling, and piping.
- Underground storage tank, electrical equipment, and switchgear house

At selected pumping stations, a switchgear house was utilized to interface the generator with the existing electrical system. The switchgear house is also designed to meet local seismic requirements. The generator can also be started from the switchgear house. The switchgear house also contained a spare breaker to feed an outdoor high service pump located on the reservoir. This pump would serve as an emergency pump in case there was damage to the pump building and the indoor high service pumps could not be utilized.

Figure 4 Typical Switchgear

Fortunately, no significant earthquakes have occurred in the Memphis area. However, Memphis has experienced several other hazards over the past 15 years. Memphis experienced an ice storm in 1994, hurricane force winds in 2003, a tornado in 2008, and straight-line winds and a tornado in 2009. During the 2003 incident, several of the pumping stations lost power for several days but the emergency generators allowed them to remain operational. Therefore, the emergency generators are not only useful in case of a seismic event but also if another hazard event occurs.

Water Piping System

Memphis Light Gas and Water owns and maintains its water distribution piping system. MLGW has approximately 20,200,000 ft. of water main. Memphis has a unique water system in that our water is obtained from an artesian source rather than surface water. The water is pumped from wells through eight major water pumping stations/treatment facilities and then through a highly interconnected distribution system to deliver water to our customers. Our water main ranges in size from 2” – 36” and various materials including PVC, PE, copper, cast iron, asbestos cement, galvanized steel, concrete, and ductile iron in the system.
Through a system wide seismic vulnerability study, it was determined that pipeline damage would depend on many factors including levels of ground shaking, soil conditions, and pipe characteristics such as materials, size, joint types, and installation conditions. In considering the damage, it was noted there were several factors to consider:

- Pipe Damage estimates are necessarily probabilistic in nature.
- The specific locations where pipes break in any given earthquake cannot be predicted, although geographic areas, such as those with a high potential for liquefaction, where a higher rate of damage is expected can be determined.

- Pipe damages estimated are subject to considerable uncertainty and numerical estimates must be interpreted simply and cautiously as the best available estimates of the approximate number of repairs to be expected.

The current methods for estimating earthquake damage to pipeline systems on a regional scale rely largely upon observed rates of damage in past earthquakes. The current state of the art provides an estimated rate of damage per length of pipe and cannot be used to identify specific locations where pipeline damage will occur, although areas with higher expected numbers of breaks can be identified (e.g., areas subject to liquefaction). These methods prove useful only for large pipeline inventories. Considering all the factors and the inherent variability in earthquake damage phenomena necessarily leads to considerable uncertainty in the regional damage estimate. The assessment was based on the vulnerability of MLGW's pipelines to earthquakes in three damage modes:

- Wave propagation (ground motion),
- Permanent ground displacement (liquefaction, settlement, lateral spreading) and,
- Bridge failures.

The inventory of MLGW's water pipelines for assessing scenario damage was extracted from the Facility Information system (FIS) database. A tabular summary of the MLGW pipeline inventory used to estimate earthquake damage is provided in Table I.

TableI

Summary of MLGW Water Pipe Inventory by Miles of Pipe

CATEGORY	PIPE SIZE (inches)								TOTAL
	2 or less	2.5 - 4	6 - 8	10 - 12	14 - 18	20 - 24	26 - 30	Greater than 30	
CI/DI	95.9	47.0	2728.4	936.9	172.6	156.3	60.4	8.3	4205.6
PVC	82.0	0.6	5.0	4.9	0.4	0	0	0	92.8
UNSPEC	2.8	3.4	6.7	1.0	0	0	0	0	13.9
AC	0	0	5.1	0	0	0	0	0	5.1
CU	3.5	0	0	0	0	0	0	0	3.5
STEEL	1.5	0	0.4	0.5	0.2	0.7	0	0	3.3
GAL	2.3	0	0	0	0	0	0	0	2.3
TOTAL	188.0	50.9	2745.7	943.3	173.2	157.0	60.4	8.3	4326.6

Pipeline damage estimates for the three earthquake scenarios were computed for all pipelines with a size of 12 inches or larger. The results of these computations are provided in Table II.

TableII

Summary of Water Pipeline Repair Estimates for Three Earthquake Scenarios

	M6.2			M7.0			M7.7		
	ALL	12+	10-	ALL	12+	10-	ALL	12+	10-
CI/DI	174	41	133	434	103	332	1293	329	963
PVC	2.8	0.0	2.8	7.2	0.0	7.1	25	0.1	25
UNSPEC	0.6	0.0	0.6	1.5	0.0	1.5	5.3	0.1	5.2
AC	0.2	0.0	0.2	0.6	0.0	0.6	2.0	0.0	2.0
CU	0.2	0.0	0.2	0.6	0.0	0.6	2.1	0.0	2.1
STEEL	0.1	0.0	0.0	0.1	0.0	0.0	0.8	0.5	0.3
GAL	0.2	0.0	0.2	0.4	0.0	0.4	1.1	0.0	1.1
Total Non-Service Repairs	180	40	140	440	100	340	1330	330	1000
Services	35	--	--	90	--	--	265	--	--
Total All Repairs	215			530			1595		

Notes:

1. 12+ indicates pipe size of 12 inches or larger
2. 10- indicates pipe size of 10 inches or less
3. Total non-service repairs and service repairs rounded to nearest 10 and nearest 5, respectively.

The distribution of estimated repairs is primarily related to the amount of distribution pipe within the area, since the variation in ground motion doesn't vary dramatically between areas within Shelby County.

For assessing pipeline damage for pipelines on bridges the HAZUS method was used. The HAZUS methodology is straightforward and is directly tied to bridge descriptors contained in the National Bridge Inventory. For our study, either extensive or complete damage are considered credible events leading to pipeline damage. A summary of bridges with water pipelines was compiled and is shown in the Figure 5 below.

These bridge crossing were further broken down into "Key" bridge crossings. "Key" bridges carrying water pipelines were considered to be those bridges with a pipeline of 12 inches or greater. Based on our information there are 58 key bridge crossings out of 149 bridges with water pipelines. Of those 58 key bridges, 28 were determined to be short span bridges based on visible details obtained through Google Earth.

Locations of Water Pipeline Bridge Crossings

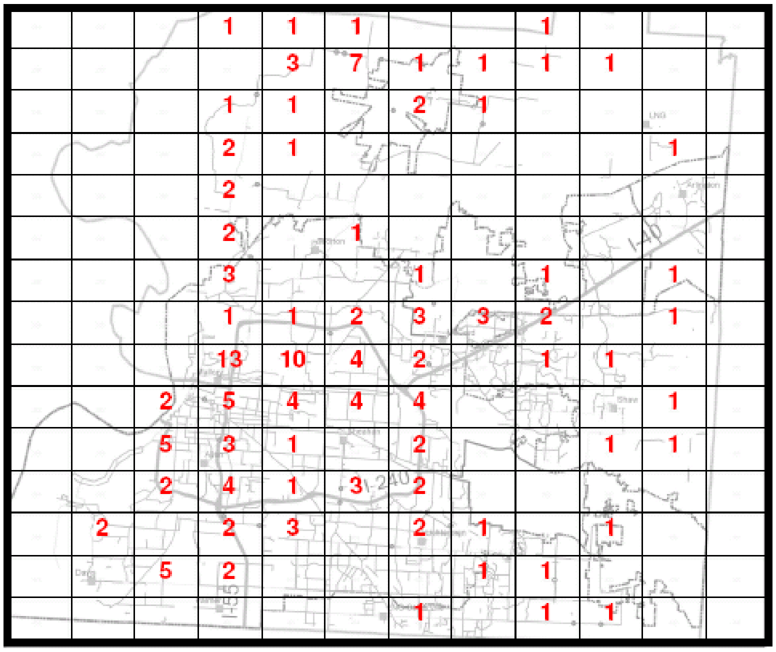


Figure 5

Figure 6 Typical Bridge Crossing

REFERENCES

[1]. MLGW Seismic Mitigation Plan 2003 for Earthquake Preparedness of Critical Water System Facilities.
[2]. MLGW Multi-Hazard Risk Assessment, Seismic Performance Objectives, March 2, 2007.
[3]. PolyProcessing Company Catalog, The High Performance Tank Resource Manual, www.polyprocessing.com.

Aged-Pipeline Renewal Plan with Enhancing Seismic Upgrade - In Case of Bureau of Public Enterprises, Ibaraki Prefecture

Hayato Nakazono, Kinya Kataishi, Shuzo Furukawa and Minoru Ikei

ABSTRACT

There are totally 1,300km pipelines held by Bureau of Public Enterprises, Ibaraki Prefecture (here in after referred to Bureau). Some pipelines have already pasted the lifetime and aged-pipelines have been increasing. On the other hand, the ordinance No.60 was promulgated by Ministry of Health, Labour and Welfare in April 2008 to ensure the protection of water supply functions in case of an earthquake that has frequently happened. In necessity of replacing aged-pipelines and enhancing seismic upgrade, the renewal plan was developed in the targeted pipelines held by Bureau. In target of this plan, the pipelines were focused in 4 regional water supplying authorities and 5 regional industrial water supplying authorities managed by Bureau. For development of the renewal plan, 2 indicators were used to evaluate. The 1st one is the deterioration indicator for evaluating the physical condition of the pipelines by the forecast formula presented by Japan Water Works Association. The 2nd one is the indicator of the important pipeline route for evaluating both the pipeline-damaged ratio and water volume affected by water-cut. Finally, the effectiveness of the investment for the renewal plan was clarified by cost-effectiveness analysis. The plan should be contributed to the sustainable water facility throughout the methodology.

Hayato Nakazono, Engineer, Group of Eastern Japan, Department of Water Supply, Tokyo Engineering Consultants Co., Ltd. , Fuji-Building, 7-4 Kasumigaseki 3-Chome, Chiyoda-Ku, Tokyo, 100-0013, Japan.
Kinya Kataishi, General Manager, Department of Water Supply, Tokyo Engineering Consultants Co., Ltd. , Fuji-Building, 7-4 Kasumigaseki 3-Chome, Chiyoda-Ku, Tokyo, 100-0013, Japan
Shuzo Furukawa, Team Leader, Group of Eastern Japan, Department of Water Supply, Tokyo Engineering Consultants Co., Ltd. , Fuji-Building, 7-4 Kasumigaseki 3-Chome, Chiyoda-Ku, Tokyo, 100-0013, Japan.
Minoru Ikei, Engineer, Overseas Department, Tokyo Engineering Consultants Co., Ltd. , Fuji-Building, 7-4 Kasumigaseki 3-Chome, Chiyoda-Ku, Tokyo, 100-0013, Japan

1. INTRODUCTION

In 17th January 1995, Great Hanshin-Awaji Earthquake has taken an enormous toll to the water supply facilities. In response to this knowledge, Bureau of Public Enterprises, Ibaraki Prefecture (here in after referred to Bureau) has reviewed the earthquake countermeasures from the viewpoint of enhancing seismic upgrade of the facilities and emergency rehabilitation system from 1996 to 1998.

There are totally 1,300km pipelines held by Bureau. Some pipelines have already pasted the lifetime and the number of aged-pipelines has been increasing. On the other hand, the ordinance No.60 was promulgated by Ministry of Health, Labour and Welfare in April 2008 to ensure the protection of water supply functions in case of an earthquake that has frequently happened. In necessity of replacing aged-pipelines and enhancing seismic upgrade, the renewal plan was developed in the targeted pipelines held by Bureau.

2. OUTLINE OF BUREAU OF PUBLIC ENTERPRISES, IBARAKI PREFECTURE

Ibaraki Prefecture is located in the central are of Japanese archipelago and the distance from Tokyo is nearly 40km. The population in Ibaraki prefecture is 2,964,000 people in the 11th place of Japanese 47 prefectures.

There are 4 regional water supplying authorities (here in after referred to RWSA) and 5 regional industrial water supplying authorities (here in after referred to RIWSA) managed by Bureau. Outlines of the RWSAs and the RIWSAs are shown in Table 1, Table 2 and Figure1. In target of this plan, the pipelines were focused in all 4 RWSAs and all 5 RIWSAs managed by Bureau.

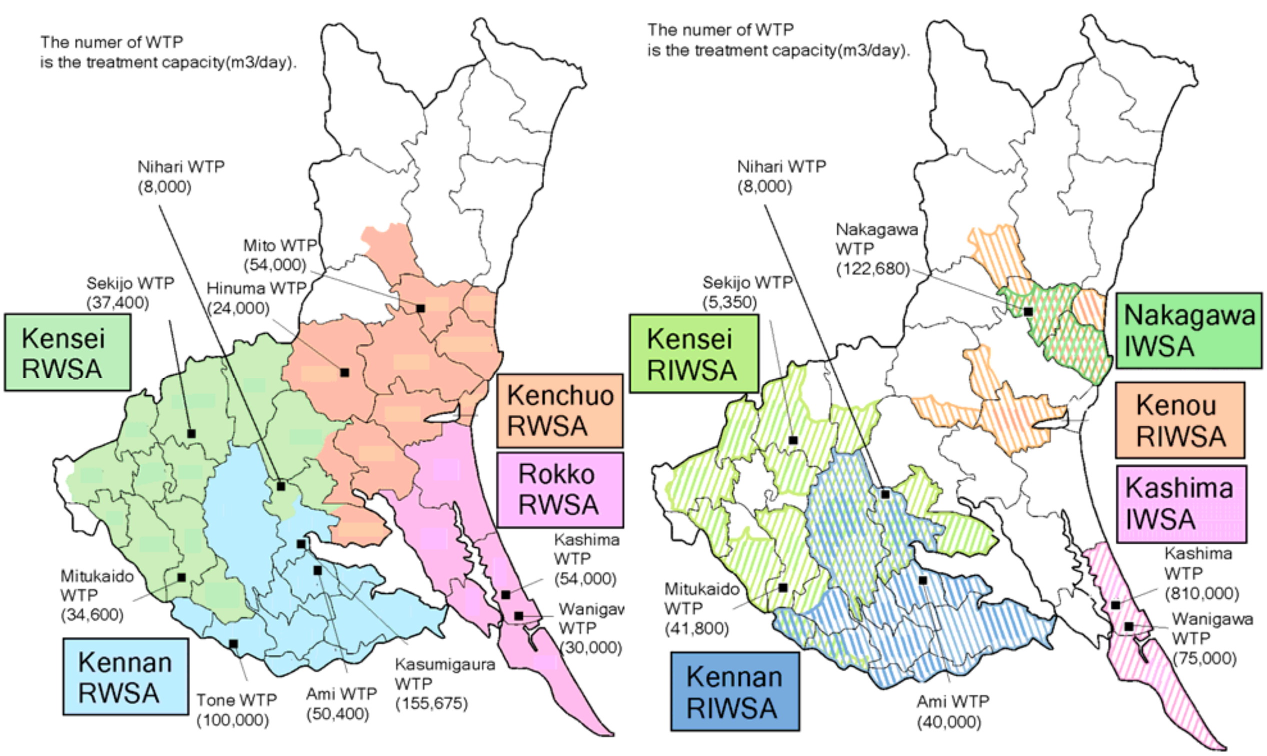


Figure 1. SUPPLY AREA OF THE RWSAs AND THE RIWSAs

Table 1. OUTLINE OF THE RWSAs

Item	Unit	Kennan RWSA[*1]	Rokko RWSA	Kensei RWSA	Kenchuo RWSA	Total
Targeted Supply Commue	—	8 MUNIs[*2] 1WSU[*3]	5 MUNIs	13 MUNIs	10 MUNIs 1WSU	34 MUNIs 2WSUs
Maximum Supply Amount	(m3)	306,075	84,000	80,000	78,000	548,075
Water Source	—	Kasumigaura (Lake), Groundwater, Tone River	Kitaura (Lake), Wani River	Kasumigaura (Lake), Kinu River, Tone River	Naka River, Hinuma River	—
Planning Supply Population	(ppl)	661,500	293,680	501,200	931,300	2,387,680
Start Year to Supply	—	Dec. 1960	Aug. 1968	Apr. 1988	Jan. 1992	—
Construction Term (Reconstruction)	—	1957 - 2012 (2004 - 2011)	1966 - 2014 (1998 - 2008)	1980 - 2011	1985 - 2011	—
Cost of Construction Term (Reconstruction)	(MM[*4] Yen)	62,229 (20,000)	42,013 (3,524)	44,400	84,583	233,225
Cost of Water Source	(MM Yen)	60,416	10,671	36,168	20,878	128,133

*1: Regional Water Supplying Authority, *2: Municipality, *3: Water Supply Union, *4: Millions

Table 2. OUTLINE OF THE RIWSAs

Item	Unit	Kennan RIWSA[*5]	Kashima IWSA[*6]	Kensei RIWSA	Nakagawa IWSA	Kenou RIWSA	Total
Targeted Supply Area	—	5 MUNIs	2 MUNIs	13 MUNIs	2 MUNIs	3 MUNIs	22 MUNIs
Targeted Supply User	—	51 Companies	66 Companies	138 Companies	6 Companies	14 Companies	275 Companies
Maximum Supply Amount	(m3)	40,000	885,000	79,650	76,680	46,000	1,127,330
Water Source	—	Kasumigaura (Lake)	Kitaura (Lake), Wani River, Groundwater	Kasumigaura (Lake), Kokai River	Naka River	Naka River, Hinuma River	—
Start Year to Supply	—	Jul. 1997	Feb. 1967	Apr. 1988	Oct. 1966	Oct. 2001	—
Construction Term (Reconstruction)	—	1985 - 2012	1966 - 1994 (1998 - 2008)	1980 - 2012	1962 - 1995 (1996 - 2002)	1995 - 2010	—
Cost of Construction Term (Reconstruction)	(MM Yen)	38,863	32,900 (17,830)	25,727	4,024 (3,073)	18,905	120,419 (20,903)
Cost of Water Source	(MM Yen)	7,896	93,918	18,533	418	5,128	125,891

*5: Regional Industrial Water Supplying Authority, *6: Industrial Water Supplying Authority

3. TARGETED PIPELINES

In 2008, there were 31km pipelines (DIP, SP) that 40 years have gone by and 233km that have already had seismic capacity. For this evaluation, the earthquake-resistance pipeline, which has the seismic capacity, was defined as K-type DIP in cohesive soil, SP and anti-earthquake DIP. The range of diameter was from 75 to 2100mm and the types of pipelines were steel pipe (here in after referred to SP) and ductile iron pipe (here in after referred to DIP). In addition, the liquefiable condition in the pipelines' installed site was clarified by using the 879 boring log datum in Ibaraki prefecture. The liquefaction phenomenon is one that ground liquefies due to stiffness degradation caused by earthquake, and the phenomenon has great impacts on the pipelines, for example, dropping out of pipe-joint and breakage of the pipe caused by stress concentration.

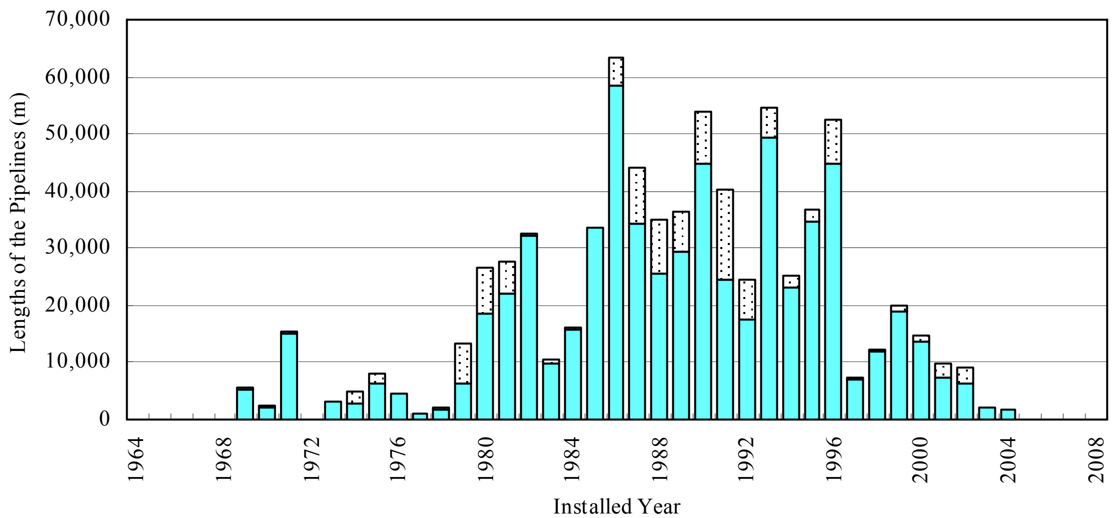


Figure 2. LENGTHS OF THE PIPELINES BY INSTALLED YEAR CONCERNING THE RWSAs

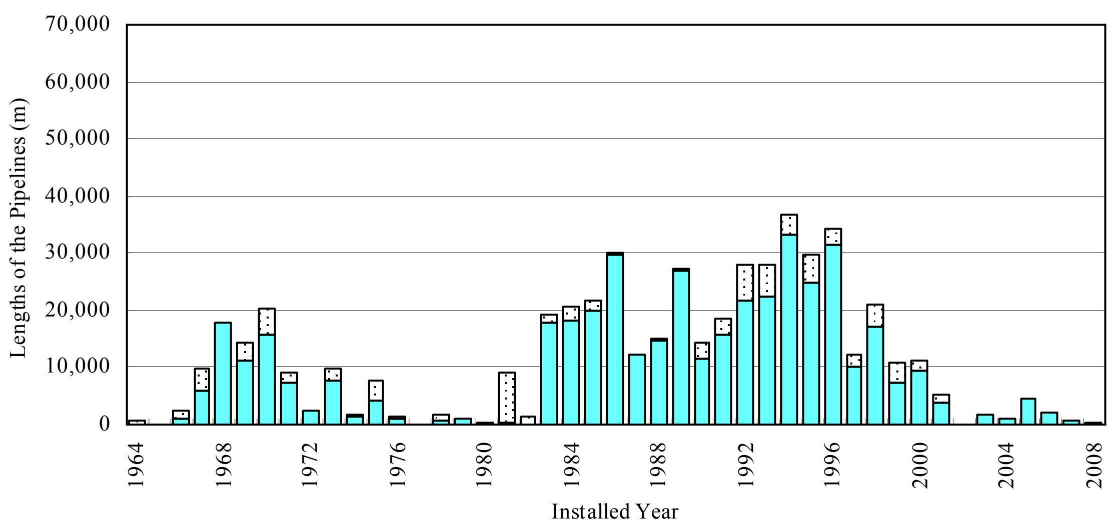


Figure 3. LENGTHS OF THE PIPELINES BY INSTALLED YEAR CONCERNING THE RIWSAs

Table 3. LENGTHS OF THE PIPELINES BY INSTALLED YEAR

Installed year	Lengths of the pipelines in each RWSA (m)					Lengths of the pipelines in each RIWSA (m)				
	Kennan RWSA	Rokko RWSA	Kensei RWSA	Kenchuo RWSA	Total	Kennan RIWSA	Kashima IWSA	Kensei RIWSA	Nakagawa IWSA & Kenou RIWSA	Total
-1968							30,154		884	31,038
1969 - 1978	9,846	37,180			47,026		68,798			68,798
1979 - 1988	110,829	5,178	107,184	79,376	302,568		19,857	97,556	13,865	131,278
1989 - 1998	36,247	79,727	118,146	109,645	343,765	81,132	12,747	133,146	23,571	250,596
1999 -	13,100	26,300	17,826		57,226		8,061	6,678	23,577	38,316
TOTAL	170,023	148,385	243,156	189,021	750,585	81,132	139,617	237,381	61,897	520,027

Table 4. LENGTHS OF THE PIPELINES BY DIAMETER

Diameter	Lengths of the pipelines in each RWSA (m)					Lengths of the pipelines in each RIWSA (m)				
	Kennan RWSA	Rokko RWSA	Kensei RWSA	Kenchuo RWSA	Total	Kennan RIWSA	Kashima IWSA	Kensei RIWSA	Nakagawa IWSA & Kenou RIWSA	Total
- φ250	14,175	11,542	77,585	26,051	129,353	17,411	16,571	110,463	1,500	145,945
φ300 - φ500	61,505	99,201	145,933	89,313	395,952	17,959	36,973	95,368	30,491	180,791
φ600 - φ800	61,367	37,051	18,374	14,937	131,729	18,884	17,158	30,330	19,823	86,195
φ900 - φ1100	15,437		1,264	39,756	56,457	19,185	19,907	1,220	6,066	46,378
φ1200 - φ1350	17,539			18,964	36,503	7,693	7,142		4,016	18,852
φ1500 - φ1650							14,635			14,635
φ1800 -		591			591		27,231			27,231
TOTAL	170,023	148,385	243,156	189,021	750,585	81,132	139,617	237,381	61,897	520,027

Table 5. LENGTHS OF THE PIPELINES BY TYPES OF PIPELINES

Type	Lengths of the pipelines in each RWSA (m)					Lengths of the pipelines in each RIWSA (m)				
	Kennan RWSA	Rokko RWSA	Kensei RWSA	Kenchuo RWSA	Total	Kennan RIWSA	Kashima IWSA	Kensei RIWSA	Nakagawa IWSA & Kenou RIWSA	Total
SP	39,050	3,092	3,090	69,939	115,170	26,034	35,163	3,914	17,699	82,809
DIP-A/T	28,914	51,352	96,615	38,342	215,223	18,361	30,614	161,434	18,571	228,981
DIP-K	100,532	89,725	143,451	79,640	413,348	33,641	73,840	71,204	24,867	203,551
DIP-S_{II} /KF	1,527	4,216		1,100	6,844	3,097		830	760	4,686
TOTAL	170,023	148,385	243,156	189,021	750,585	81,132	139,617	237,381	61,897	520,027

Table 6. LENGTHS OF THE PIPELINES BY LIQUEFACTION ASSESSMENT

Liquefaction Assessment (Probability of Occurrence)	Lengths of the pipelines in each RWSA (m)					Lengths of the pipelines in each RIWSA (m)				
	Kennan RWSA	Rokko RWSA	Kensei RWSA	Kenchuo RWSA	Total	Kennan RIWSA	Kashima IWSA	Kensei RIWSA	Nakagawa IWSA & Kenou RIWSA	Total
QUITE-HIGH	17,388	16,131	37,510	9,450	80,479	11,061	36,617	50,013	333	98,024
HIGH	57,099	36,853	113,869	42,142	249,963	18,090	101,933	78,206	25,020	223,249
LOW	95,535	95,401	91,777	137,429	420,142	51,981	1,067	109,162	36,544	198,754
TOTAL	170,023	148,385	243,156	189,021	750,585	81,132	139,617	237,381	61,897	520,027

4. METHODOLOGY FOR RENEWAL PLANNING

For development of the renewal plan, 2 indicators were used to evaluate. The 1st one is the deterioration indicator for evaluating the physical condition of the pipelines by the forecast formula [1] presented by Japan Water Works Association (here in after referred to JWWA). The 2nd one is the indicator of the important pipeline route for evaluating both the pipeline-damaged ratio and water volume affected by water-cut. The assumption of earthquake ground motion was substituted the data of National Seismic Hazard Maps[2] by the Headquarters for Earthquake Research Promotion for and the pipelines-damaged ratio was calculated by the formula of damaged pipeline[3] under earthquake presented by JWWA.

Table 7. EVALUATION INDICATORS

Classification		Evaluation Items
Deterioration Indicator	Physical Condition	・Elapsed years ・Risk of trouble by types of pipeline ・Leakage of pipeline ・Inside lining applications ・Soil conditions ・Pipeline materials
Indicator of the Important Pipeline Route		・pipeline-damaged ratio
		・water volume affected by water-cut

5. APPLICATION OF METHODOLOGY

The evaluation results for this methodology are shown in Figure 4, vertical axis as deterioration indicator and horizontal axis as indicator of the important pipeline route. In present situation, all of the pipelines were evaluated as the finest rank from the viewpoint of the deterioration indicator. Through the targeted pipelines are fine in 2008, the renewal term of the aged-pipeline will be definitely coming in the future. The renewal term from the deterioration, which has been back-calculated as the time for becoming the rank: “the implementation of the renewal construction”, was indentified as the deterioration indicator. In addition, in this study, the aged-pipeline was defined as the elapsed 65 years for DIP and as the elapsed 55 years for SP, based on the average value of the renewal term from the deterioration.

The renewal time from the deterioration was quite-variable as shown Figure 4. Specifically, the renewal time was dispersed during 2025 to 2070 and concentrated before and after 2048. As a result, to concentrate on the high demands for the renewal of the pipelines before and after 2048, the time was equalized by the priority of the renewal pipelines.

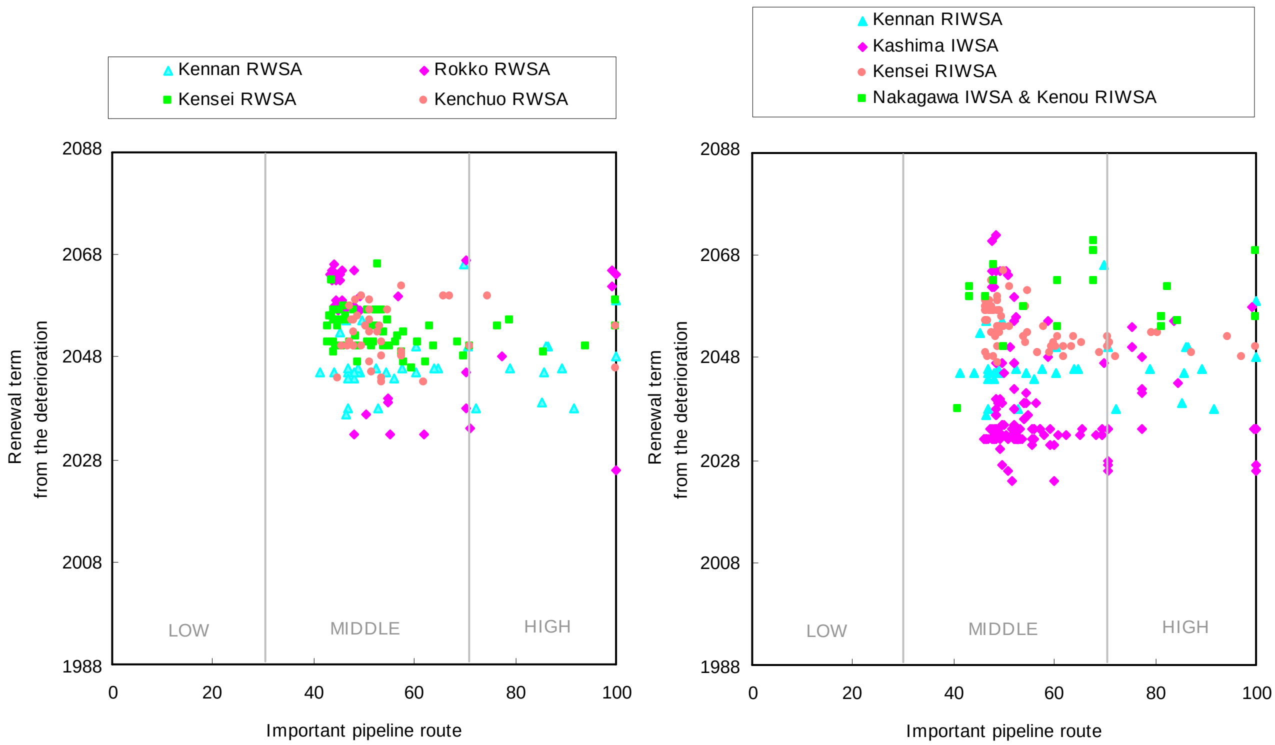


Figure 4. EVALUATION RESULTS FOR THE METHODOLOGY

6. DETERMINATION OF RENEWAL PIPELINES' PRIORITY

Based on the methodology for renewal planning, the priority of the renewal pipelines was decided by 2 indicators. The renewal time of aged-pipelines was set in the end of the lifetime as the elapsed 65 years for DIP and as the elapsed 55 years for SP, and the renewal time for enhancing seismic upgrade was set before the-end-of-life by using the priority of the renewal pipelines in order to increase the earthquake resistance ratio. As a result, the renewal term for enhancing seismic upgrade was set during 2010 to 2033 concerning the regional water supplying authorities and during 2010 to 2049 concerning the regional industrial water supplying authorities. Furthermore, considering the other construction cost, the renewal time for enhancing seismic upgrade was set not to have great impact to the financial condition of the Bureau.

The ratio and lengths of the pipelines by quinquennial renewal installed term are shown in Table 8, 9 and Figure 5, 6. Main determinant, concerning the renewal time for enhancing seismic upgrade, is the other construction cost. In the regional water supplying authorities, it is necessary to carry out the renewal installation of more than 100km during 5 years.

Table 8. RATIO BY RENEWAL INSTALLED TERM CONCERNING THE RWSAs

Item	Unit	2008	2010 -2014	2015 -2019	2020 -2024	2025 -2029	2030 -2034	2035 -2039	2040 -2044	2045 -2049	2050 -2054	2055 -2059	2060 -2064	2065 -2069	2070 -2074
Ratio for renewing aged-pipelines	(%)	-	-	-	-	0	3	12	21	48	74	90	98	100	100
Ratio for enhancing seismic upgrade	(%)	-	7	33	61	86	100	100	100	100	100	100	100	100	100
Earthquake resistance ratio	(%)	46	50	64	79	93	100	100	100	100	100	100	100	100	100

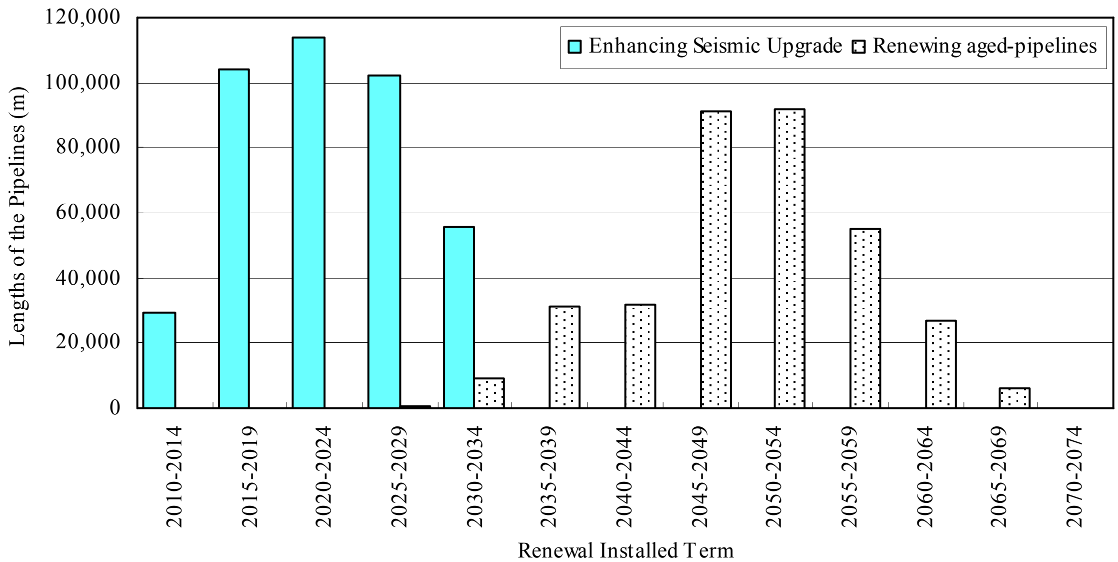


Figure 5. LENGTHS OF THE PIPELINES BY RENEWAL INSTALLED TERM CONCERNING THE RWSAs

Table 9. RATIO BY RENEWAL INSTALLED TERM CONCERNING THE RIWSAs

Item	Unit	2008	2010 -2014	2015 -2019	2020 -2024	2025 -2029	2030 -2034	2035 -2039	2040 -2044	2045 -2049	2050 -2054	2055 -2059	2060 -2064	2065 -2069	2070 -2074
Ratio for renewing aged-pipelines	(%)	-	-	-	4	11	15	24	28	45	66	85	100	100	100
Ratio for enhancing seismic upgrade	(%)	-	6	23	31	38	46	52	77	100	100	100	100	100	100
Earthquake resistance ratio	(%)	29	33	45	52	56	62	66	84	100	100	100	100	100	100

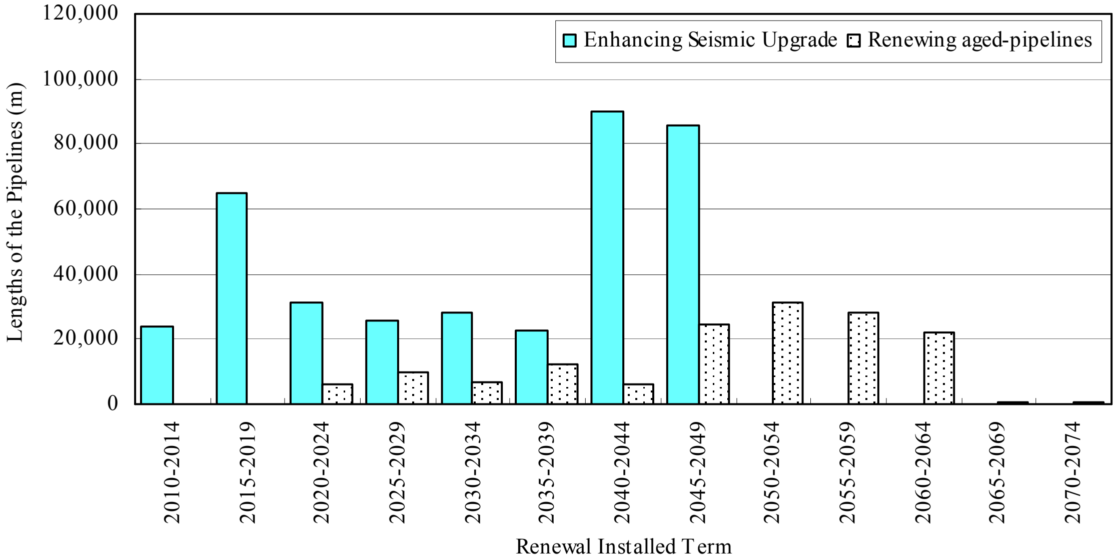


Figure 6. LENGTHS OF THE PIPELINES BY RENEWAL INSTALLED TREM CONCERNING THE RIWSAs

7. COST-BENEFIT ANALYSIS

The effectiveness of the investment for the renewal plan was clarified by cost-benefit analysis of water supply presented by Ministry of Health, Labour and Welfare [4] and of industrial water supply presented by Ministry of International Trade and Industry [5]. Indicators and results for cost-benefit analysis, concerning the RWSAs and the RIWSAs, are shown in Figure 10. The Criterion for the judgment of the investment's effectiveness is that the cost benefit ratio is more than 1.00 and whether the benefit for the renewal plan justifies the cost required or not.

As a result, the cost benefit ratios of RWSA and RIWSA are 5.01 and 1.00. There is the effectiveness of the investment fro renewal plan concerning both RWSA and RIWSA.

Figure 10. INDICATORS AND RESULTS FOR COST-BENEFIT ANALYSIS CONCERNING THE RWSAs AND RIWSAs

Item		Regional Water Supplying Authorities	Regional Industrial Water Supplying Authorities
Indicator for Cost		Cost for aged-pipeline renewal plan with enhancing seismic upgrade	Cost for aged-pipeline renewal plan with enhancing seismic upgrade
Indicator for Benefit	For User	Benefit for decreasing damage of water outage	Benefit for avoiding the shutdown caused by water outage
	For Supplier	Benefit for decreasing restoration work	Benefit for avoiding the risk of facilities' damage caused by aged facilities
		Benefit for decreasing water leakage	Benefit for avoiding the risk of facilities' damage caused by earthquake
		Benefit for decreasing cost of operation and maintenance	Benefit for decreasing cost of operation and maintenance
Result of Cost-Benefit Ratio		5.01	1.00

8. CONCLUSION

In changing social structures such as a shrinking and aging population, the demands for the renewal of the aged-pipeline are increasing throughout Japan. On the other hand, as of March 2007, the rate of essential pipelines with seismic capacity is approximately 12% in Japan. Once the earthquake happens, non-earthquake-resistance pipelines fall victim and have a great impact to supply purified water. The plan should be contributed to the sustainable water facility throughout the methodology and the equalization of the financial condition.

[1] Japan Water Works Association: Waterworks Facility Renovation Guidelines, 2005.05
[2] The Headquarters for Earthquake Research Promotion HP: http://www.jishin.go.jp/main/index.html
[3] Japan Water Works Association: Forecast of damaged pipeline by earthquake, 1998.11
[4] Ministry of Health, Labour and Welfare: Manual for Cost-Benefit Analysis of water supply, 2007.07
[5] Ministry of International Trade and Industry: Manual for Cost-Benefit Analysis of industrial water supply, 1999.04

Magic R: Seismic Design of Water Tanks

John Eidinger[1]

ABSTRACT

Seismic design of water tanks relies upon a number of rules issued by various code-setting groups. The AWWA code [1] includes a factor "R" that is used to establish forces for the seismic design of water tanks (circular welded steel, circular bolted steel, circular prestressed concrete, rectangular reinforced concrete, circular wood, and open cut lined with roof systems). The "R" factor is sometimes called a "ductility factor" or "response modification factor", and is often in the range of 3.5 to 4.5. Essentially, the R factor is used to adjust the elastically-computed seismic forces, $V = \frac{ZIC}{R} W$, where V = seismic base shear, Z = local site specific peak ground acceleration, I = importance factor, C = normalized response spectra ordinate, W = weight, with adjustment to suitably combine the effects of the structure, water impulsive and water convective (sloshing) components of the total load.

This paper examines the technical basis of "R". Is it from test? empirical data? experience? a desire to keep the cost of construction low? The evidence in this paper shows that the "R" factors in the code are based on "magic", that is to say, without factual evidence. When the empirical evidence is examined for more than 500 tanks and reservoirs, we find that the use of R has led to poor performance of water tanks under moderate to strong ground motions, often leading to loss of water contents.

This paper provides recommendations as to how to adjust code R values, as well as refinements in detailing for side entry pipes, bottom entry pipes, and the roof. These recommendations are made in reflection of the observed empirical evidence of actual damage of tanks in past earthquakes, tempered with findings from shake table test data. By adopting these refinements, it is hoped to achieve cost effective seismic design of water tanks that also provides high confidence of suitably reliable performance in large earthquakes.

SEISMIC DESIGN CODES FOR BUILDINGS

Ductility plays an important role in the response of structures due to earthquake motions. Prior to the mid-1980s, the common code approach to seismic design for regular buildings (not tanks) in high seismic areas of California) was as follows:

1933 to 1943 (Los Angeles)

V = 0.02W to V = 0.10W, with the base shear coefficient (0.02 to 0.10) chosen depending on the type of building.

1943 to 1957 (Los Angeles)

Taller buildings, being more flexible, were allowed to be designed with lower base shears.

---

[1] John Eidinger, President, G&E Engineering Systems, eidinger@geEngineeringSystems.com

$$V = \frac{0.6}{N+4.5}W,$$

where N = number of floors.

Sample: N = 1, then V = 0.133W, or if N = 5, then V = 0.063W

1956 to 1974 (San Francisco)

$$V = \frac{K}{T}W,$$

where K = 0.035 for non-building structures and T = period of the structure in seconds, and K/T (max) = 0.10.

1975 to 2009 (Modern Era)

Since about 1975, almost all building codes in the USA have been reformulated to compute required seismic base shear as follows:

$$V = \frac{PGA * I * C}{R}W$$

where PGA = design level horizontal peak ground acceleration, set at the 475 year motion, or 2/3 of 2,475 year motion, I = importance factor (I= 1 for regular buildings, 1.25 for important buildings or 1.5 for critical buildings), C = response spectral coefficient for 5% damped spectra (usually about 2.75 for structures at the peak of the spectra), and W = dead weight of the building, sometimes including a percentage of live load). In this formulation, R includes the effects of hysteretic energy from yielding, increased damping over 5%, and all other factors of safety embedded into the code design approach. For working stress design approaches, R is replaced with Rw; for ultimate strength design approaches, R is often set at R = Rw / 1.4, just enough to offset the load factors used in the design approach. Depending on which code is considered, Rw values have ranged from 1.5 (for unreinforced masonry construction, where allowed) to as high as 12 (for presumed ductile steel moment frame buildings). The 1997 UBC provided R = 8.5 (same as Rw = 12) for special moment frame steel buildings.

SEISMIC DESIGN FOR WATER TANKS

Two of the earliest "codes" or manuals or practice for fluid-filled containers are work by Housner [2, 1954] and TID 7024 [3, 1963]. These approaches assumed that R = 1, and assumed the tank is rigid (responds at PGA) for the impulsive mode. The net result was that V = 0.25*W for small tanks (for radius of tank = 13 feet, height of water = 15 feet). The convective mode was calculated elastically (R=1) and combined with the impulsive mode by absolute sum. The long period of the convective mode (commonly T = 3 to 8 seconds) as compared to the high frequency of the impulsive mode (f = 3 to 8 hertz) strongly suggested that the maximum impulsive forces could occur at (or nearly at) the time of maximum convective forces, and hence an absolute sum of the two terms seemed reasonable. TID 7024 required that a ring girder be placed at the top level of the tank shell "to provide stability against excessive distortion due to the lateral forces generated by the accelerated fluid". TID 7024 specifically allowed that sloshing forces need not be accommodated in the design if damage to the rood was considered acceptable. TID 7024 recognized that uplift of tanks shells promoted higher stressed in compression, leading to increased chance of damage due to wall buckling (elephant foot).

By about 1970, it was recognized that the impulsive mode of most common-sized water tanks was in the range of f = 3 to 8 hertz, and so the seismic base shear from the impulsive mode should be computed using the amplified spectral coordinate. For design of water tanks outside of the nuclear industry, it was also decided (largely for convenience) to base the design spectra using horizontal 5%-damped spectra, as that was the default set in regular building codes. In the nuclear industry, it was commonly set that the impulsive mode for steel tanks had 2% damping, and the convective (sloshing) mode had 0.5% damping.

By the mid-1990s, various AWWA code committees diverged on R values. The D100 code (for steel tanks) allowed that the base shear and slosh height in the convective mode could be computed buy dividing by "R"; whereas the D110 code (for concrete tanks) the R value for the convective mode is 1. In some codes, the impulsive mode and convective mode base shears could be combined by square-root of the sum or the squares (although there is little technical basis to support this). Some practitioners further divided the slosh height by R, a practice that could be interpreted as acceptable by code, but that has no technical basis (in other words, the wave heights are not affected in any appreciable manner by any local yielding in the steel shell).

In 1978, a non-mandatory seismic design code was issued for water storage tanks. By non-mandatory, the code was optional for seismic zones 1, 2 and 3, but require in seismic zone 4. By "seismic zones", zone 4 was limited to areas of the USA with PGA = 0.4g (or higher); zone 3 was for areas with PGA = 0.3g, zone 2 with PGA = 0.15g, zone 1 with PGA = 0.075g, and zone 0 was for non-seismic areas.

$$V = ZK\left(0.14\left(W_{Shell} + W_{Roof} + W_{Water-Impulsive}\right) + C_1 S W_{Water-Sloshing}\right)$$

with

Z = 1 (zone 4), 0.75 (zone 3), 0.375 (zone 2), 0.1875 (zone 1)

K = 2.00 (anchored flat bottom tank) or 2.50 (unanchored flat bottom tank)

S = 1.0 (rock site), 1.2 (stiff soil site), 1.5 (soft sol site), and CS $\leq$0.14

For an anchored tank on rock (D = 140 feet, H = 40 feet) with T (impulsive) = 0.2 seconds and T (sloshing) = 7.7 seconds and located in zone 4 on a rock site, then

V = (1.0)(2.0)(0.14W(steel + water impulsive) + 0.013W(sloshing))

For a moderately large 4.6 MG tank with D = 140 feet and H = 40 feet, built with mild steel (Fy = 30 ksi) with average wall t = 0.45 inches, average roof t = 0.1875 inches, then the weight of the steel is 441,000 pounds, the weight of water (when full) is 38,423,000 pounds. The weight of the contents (water) is 87 times more than the weight of the steel in this tank. For this tank, the weight of water in the sloshing (convective) mode is about 23,438,000 pounds, and the weight of water in the impulsive mode is about 12,700,000 pounds. Thus, for this tank, the total base shear is V = 3,679,000 pounds (impulsive) + 610,000 pounds (sloshing) = 4,289,000 pounds (total), or V = 0.110W. If the tank where unanchored, V = 0.138W.

In contrast, if one were to assume that the tank were to respond elastically, for a horizontal PGA = 0.40g, and assuming about 2% damping in the impulsive mode, then the elastically computed base shear would be about SA(2%, 0.2 seconds) = 1.20g, SA (2%, 7.9 seconds) = 0.08g, then V = 1.2(441,000) + 1.2(12,700,000) + 0.08(23,438,000) = 529,000 + 15,240,000 + 1,875,000 = 17,644,000 pounds, or V = 0.454W.

Examining these results, the 1978 AWWA code infers R = 0.454 / 0.110 = 4.13 (anchored) or R = 0.454 / 0.138 = 3.29 (unanchored).

This simple example ignores variations such as how the impulsive and sloshing modes should be combined (absolute sum or SRSS), higher mode effects, the code damping (commonly 5%) and the observed damping (commonly 2% for the impulsive mode and 0.5% for the convective mode). While all these variations are important, their cumulative effect is secondary as compared to the magic R effect is deciding how much base shear (and corresponding overturning moment) for which to design.

The inferred R (in the 1978 code) varies whether or not the tank is anchored or unanchored. This is a direct result of the 2.5 (unanchored) and 2.0 (anchored) multipliers that the 1978 code authors used, which was geared to *penalize* unanchored tanks (i.e., require a higher base shear force for design). Once we convert the 1978 base shear formula to the mode modern V = (ZIC/R)W formulation, we end up having to assign the energy dissipation in an anchored tank to the bolts, and energy dissipation in the unanchored tank to the uplifted sketch plate, and then observe that the common detailing of anchor bolts is non-ductile (failure in the threads), and the common detailing of sketch plates welds have a large stress riser (at the fillet welds). This is nonsense, as any beneficial yielding of the anchor bolts or sketch plates results in a trivial amount of energy absorption as compared to the mass of the water versus the available hysteretic energy absorption

The AWWA code also incorporates other serious flaws.

Once the seismic overturning moment is calculated, the code then requires that the vertical stress in the shell be less than the buckling stress (this is a good provision), as calculated using the traditional σ = M/S. For a shell annulus with D(inside) = 140 feet and t = 0.60 inches,

$S = \dfrac{\pi\left(d_{outside}^4 - d_{inside}^4\right)}{32 d_{outside}}$, and substituting d(outside) = 140x12+2*0.60 and d(inside)=140*12, we get S = 1,330,499 inches^3.

This infers that the shell of the tank behaves as a long beam, with "plane sections remaining plane". Ignoring the weight of the steel shell, the code formula for vertical stress is:

$\sigma_c = \left(\dfrac{1.273M}{D^2}\right)\dfrac{1}{12t}$, where M is in pound-feet, D in inches and t in inches.

Assuming D = 140 feet, t = 0.60 inches, and making the conversions from feet to inches, then we get the same result as above, or:

$$\sigma_c = \left(\frac{1.273 * M * 12}{(140 * 12)^2}\right)\frac{1}{12 * 0.60} = \frac{M}{1,330,275}\, psi.$$

Since the selection of the bottom course shell thickness is such a critical factor in preventing buckling, we must ask: do plane-sections-remain-plane in an at grade tank? Shake table test data performed by Akira Niwa [5] shows the answer is clearly NO for unanchored tanks, and perhaps not such a bad analogy for anchored tanks (see Section 4 for details). However, the AWWA code makes no provision for calculating the true state of stress in the tank shell due to overturning moment, a severe deficiency that perhaps is compounded by the rather arbitrary selection of R.

Another twist in the AWWA code is how the code treats the allowable stress in compression against buckling. In the 1978 code, the allowable stress in compression in the bottom course was set at 1.333 times the allowable compressive stress under dead weight, plus a factor that reflected that the hoop tensile stress due to internal water pressure has been shown to resist the tendency to buckle the shell due to vertical compression (only half this effect is allowed): $\sigma_{eq} = 1.333\left(\sigma_{allow} + \frac{\Delta\sigma_{cr}}{2}\right)$, and $\Delta\sigma_{cr} = \frac{\Delta C_c E t}{R}$. Say for our example tank, water pressure at the bottom of the tank is 40 feet * 62.4 pcf / 144 = 17.33 psi. Say E = 29,600,000 psi. R = 70 feet (radius), t = 0.60 inches, then the ΔC_c = 0.21 (based on code nomograph), and $\Delta\sigma_{cr}$ = (0.21)(29,600,000)(0.60)/(70*12) = 4,440 psi, or the total allowable compressive stress is increased by 2,220 * 1.333 = 2,959 psi, over and above the stress to safely prevent buckling (=1.333*1,395 = 1,860 psi), due to vertical stress alone (limited to yield), or a total of 4,819 psi. In the 1978 code, a warning is provided that there is controversy over this factor, stemming from the idea that the simultaneous effects of vertical earthquake could be decreasing (or increasing) the beneficial hoop tensile stress at the same time as the maximum vertical stress from overturning moment is applied.

In the 1996 and 2005 AWWA codes, this factor is further confused by the requirement that the $\Delta\sigma_{cr}$ can only be credited for unanchored tanks, but not anchored tanks. The net effect is that for the AWWA 1996 and 2005 codes, unanchored tanks are allowed to have thinner bottom course shells than for anchored tanks. The empirical evidence in Section 5 shows this to be a dubious practice. In contrast, the US NRC never allows credit for $\Delta\sigma_{cr}$, whether anchored or unanchored, as a safety precaution for commercial nuclear power plants.

The API [4] also provides for seismic design of storage tanks. The API 650 standard of 1990 is essentially identical to the AWWA code of 1978, except that an importance factor, I, is introduced. I is set to 1.0 for regular tanks, and up to 1.5 for important tanks that must provide emergency service to the public; and (K)(0.14) is replaced with 0.24 (about 15% lower than AWWA) and the API long period sloshing spectra constant is similarly about 15% lower than the AWWA value. In other words, API would allow about a 15% lower seismic load when I = 1.0; but when the engineer selects I = 1.25 (or 1.50), the API code would ultimately require a higher base shear. The API code allows for an increase in allowable shell compressive stress to account for hoop tension, but limits the total allowable vertical stress to no more than 50% Fy. For a tank with D = 140 feet and t = 0.60 inches, the allowable seismic vertical compressive stress would be 4,286 psi. In other words, the API code provides a somewhat larger factor of safety on buckling than the comparable AWWA D100 code. Of course, the true stress needed to initiate a buckle in the shell is generally much higher than either the AWWA- or API-computed values, and the ratio of the true stress (computed from elastic-plastic theory) versus the allowables provided by AWWA code is a large reason that tanks do not fail under earthquakes any more often than they already do.

TEST EVIDENCE

In 1976, Niwa [5] performed shake table tests of water-filled tanks at Berkeley. Niwa clearly showed that there is dynamic amplification of the impulsive mode: i.e., it responds according to amplified spectral acceleration, and not the PGA. In part based on these tests, the codes and practices used up to that time (Housner 1957, TID 7024 1963) had to be changed.

Niwa showed that for anchored tanks, the elastic shell stresses could be reasonably estimated using a cantilever beam model with a combination of amplified impulsive and convective modes. Niwa also showed that for unanchored tanks subject to uplift, that the rocking response is highly nonlinear, and no appropriate single-degree-of-freedom oscillator model can be used to accurately predict the response.

Niwa showed that the Housner slosh-height analog model (H = 0.42 * Sac * D, where H = slosh height, Sac = 5% damped spectra at the sloshing period, D = tank diameter) under predicts actual unrestricted slosh heights by 15 to 32 percent or so; in part, this may be due to neglecting higher mode effects of waves. The Niwa tests do not justify applying a "R" factor to reduce slosh heights or convective-induced shell stresses.

Niwa showed that the code-computed compressive stresses due to overturning moments on an anchored tank were under-or over-predicted by -18%, -7% or +64% for three different seismic input motions adjusted to achieve PGA = 0.5g input. This finding partially justifies use of an R factor of perhaps 1.13 (on average) * other factors of safety.

In Niwa's tests, the D100 allowable for buckling stress (excluding hoop effects) was 1,560 psi. Actual measured compressive stresses from several tests were as high as 3,698 psi, yet no buckling was observed. This shows at least a factor of safety of 2 on buckling if hoop pressure effects are excluded.

Key conclusions form the Niwa tests are as follows:

- Computation of overturning moments and resulting compressive stresses, using AWWA D100 simple beam analogies, is reasonably correct for anchored tanks, but entirely speculative for unanchored tanks.
- Actual shell buckling occurs at substantially higher stresses than the allowables in the D100 code.

Extrapolating the Niwa findings towards R, we observe the following:

- Unanchored tanks respond in a highly nonlinear way once they begin to uplift.
- Computation of vertical membrane stresses in unanchored tanks (subject to uplift) using simplified code formula is highly speculative. The computed stresses may be off by several hundred percent. A three-dimensional model that captures both uplift and hoop breathing modes can better predict these stresses. Computation of

vertical membrane stresses in anchored tanks is reasonably predicted using modern (post 1975) codes that include amplification of the impulsive mode.

- For anchored tanks, R = 2 (or so) would be justified to avoid initiation of tank wall buckling, using AWWA D100-2005. This R value is better described as a Factor of Safety against shell buckling, than as an energy absorption factor. Proper detailing of side entry (accommodate at least 4 inches of uplift) and bottom entry pipes (at least 2 feet from shell) is required to accommodate uplift, as uplift may still occur using the factored R loads.
- For unanchored tanks, the actual shell buckling stresses will vary based on rigid (concrete ring beam) or flexible (tank directly on grade) conditions. Higher membrane stresses are possible for tanks on concrete ring beams; lower on flexible foundations. Actual membrane stresses cannot be accurately predicted using code formulations. If an R value is to be rationally set, then it should vary based on whether or not the tank shell sits on a concrete ring beam or on a flexible asphalt-on-soil condition.

EMPIRICAL EVIDENCE

We collected actual performance data for 542 at-grade welded steel tanks for 20 earthquakes from 1933 (Long Beach) through 2003 (Paso Robles). We developed statistics for damage / no damage, sorted by anchored and unanchored steel tanks, and degree of fill at the time of the earthquake. The complete database for 532 tanks with PGA ≥ 0.10g and reduction of the data to fragilities is provided in [6]. In this report, we describe the performance of 10 steel tanks from the 2003 Paso Robles earthquake.
The damage states are as follows:

- DS 1. No damage.
- DS 2. Damage to a pipe creates only slight leaks or minor repairs (such as damage to an overflow pipe). The tank roof might be damaged. The tank remains in service after the earthquake, with relatively minor cost repairs. Leaks do not represent a credible life-safety threat due to erosion or inundation.
- DS 3. Tank wall buckling has occurred, but without leak of tank contents. The tank remains in service immediately after the earthquake. Relatively expensive repairs (or tank replacement) are performed some time after the earthquake.
- DS 4. Tank wall buckling has occurred, or side / bottom entry pipes have broken, with loss of tank contents. The tank is out of service immediately after the earthquake. Tank replacement or expensive repairs are needed to restore the tank to service. The leaking contents could present erosion or inundation risks under certain (mostly infrequent) circumstances.
- DS 5. Tank has structurally collapsed and lost all its contents. The leaking contents present erosion or inundation risks under certain (mostly infrequent) circumstances.

Table 1 provides the breakdown of the number of tanks with various damage states. (Note: one tank was in DS 5 collapsed due to collapse of an adjacent tank – this tank was removed from the database used for developing fragilities).

Table 1. Tank Database

PGA (g)	All Tanks	DS = 1	DS = 2	DS = 3	DS = 4	DS = 5
0.10	8	4	4	0	0	0
0.16	263	196	42	13	8	4
0.26	65	32	18	11	4	0
0.36	56	22	19	8	6	1
0.47	47	32	11	3	1	0
0.56	53	26	15	7	3	2
0.67	25	9	5	5	3	3
0.87	14	10	0	1	3	0
1.18	10	1	3	0	0	6
Total (542)	532	331	112	47	25	16

Effect of Fill Level

Table 2 presents fragility curves that were calculated for a variety of fill levels in the tank database.

Table 2. Fragility Curves, Tanks, As a Function of Fill Level

DS	A, g	Beta	A, g	Beta	A, g	Beta	A, g	Beta	A, g	Beta
DS≥2	0.38	0.80	0.56	0.80	0.18	0.80	0.22	0.80	0.13	0.07
DS≥3	0.86	0.80	>2.00	0.40	0.73	0.80	0.70	0.80	0.67	0.80
DS≥4	1.18	0.61			1.14	0.80	1.09	0.80	1.01	0.80
DS=5	1.16	0.07			1.16	0.40	1.16	0.41	1.15	0.10
	All Tanks N=531		Fill < 50% N=95		Fill ≥ 50% N=251		Fill ≥ 60% N=209		Fill ≥ 90% N=120	

In Table 2, "A" represents the median PGA value (in g) value to reach or exceed a particular damage state, and Beta is the lognormal standard deviation. N is the number of tanks in the particular analysis.

Effect of Anchorage

Table 3 shows that anchored tanks have performed much better than unanchored tanks.

Table 3. Fragility Curves, Tanks, As a Function of Fill Level and Anchorage (through 1994)

DS	A, g	Beta	A, g	Beta	A, g	Beta
DS≥2	0.18	0.80	0.71	0.80	0.15	0.12
DS≥3	0.73	0.80	2.36	0.80	0.62	0.80
DS≥4	1.14	0.80	3.72	0.80	1.06	0.80
DS=5	1.16	0.80	4.26	0.80	1.13	0.10
	Fill ≥ 50% All N=251		Fill ≥ 50% Anchored N=46		Fill ≥ 50% Unanchored N=205	

The performance of 10 at-grade steel tanks in the 2003 Paso Robles earthquake was as follows:

- Morro Beach. 4 tanks were unanchored bolted steel tanks, all located at one site that had PGA ~0.10g. All four of these tanks experienced slight yielding of the holes around the bolt holes at the base of the tank, resulting in very slight leaks (drips).
- Paso Robles. 1 tank was welded steel, ~0.1 MG (~25 feet diameter), unanchored without a concrete anchor ring. This tank experience PGA ~0.35 to 0.40g. The lowest course of the shell buckled severely, but did not leak; the buckle extended 360° around the tank, with permanent bulge of ~3 inches outwards on one side and ~1 inch outwards on the opposite side. During the earthquake, the buckle was wider and deeper, but rebounded somewhat after the end of shaking. Four of four side entry pipes broke (none had flexible connectors). The tank owner repaired the tank by fixing all the side entry pipes, but left the buckled lower course in place.
- Paso Robles. 2 tanks were welded steel, unanchored on concrete ring beams, each 4 MG (132 feet diameter, 41.6 feet high). The newer tank was designed per AWWA D100 in 2001. The older tank was built circa 1970, with uncertain design basis. Site motion was about PGA = 0.30g to 0.35g. Both tanks had uplift, likely of comparable amounts (2 to 4 inches) based on observations of damaged pipes. The older uplifted tank yielded its bottom course and bottom plate (as evidenced by flaked-off paint). Two side entry pipes in the newer tank broke (one had a Dresser Coupling hat exceeded its limit and pulled its gaskets; the other broke underground at a megalug joint, as the uplifted tank put excessive tension forces on the megalug causing it to slip. On the older tank, a similar megalug connection did not break The interior bottom-entry pipe for the older tank tore where connected to the bottom plate (it was only about 18 inches from the exterior wall that uplifted), and the tank leaked all its contents. Many of the roof-level steel channels holding up the steel roof on the older tank twisted when the tank uplifted; the channels disengaged from the roof; comparable roof-level damage was not observed in the newer tank.
- Templeton. 3 steel tanks were located at the same site, likely exposed to PGA between 0.25g to 0.30g. One was at-grade bolted steel, unanchored. One was at-grade welded steel, unanchored. One was at-grade welded steel, anchored and with all side entry pipes having flexible connectors. All three tanks were 0.1MG to 1.0 MG. The at-grade bolted steel tank had elephant foot buckling with minor leaks. The at-grade unanchored welded tank uplifted several inches and damaged a bottom level drain line, and had severely damaged exterior attached electrical cables. The anchored steel tank had no observable damage.

As seen in Table 3, the empirical evidence for the benefits of anchored tanks is clear. The median PGA value to reach various damage states is about 3 to 4 times higher for anchored tanks as for unanchored tanks. It should be noted, however, that the anchored tank database (N=46) is much smaller than the unanchored tank database (N=251), and fill levels may not have been known for all tanks in the anchored tank database. The empirical evidence strongly suggests that anchored tanks outperform unanchored tanks.

The empirical data suggests that the 1996 AWWA D100 code change to penalize anchored tanks (still reflected in the 2005 code) is probably unwarranted.

CONCLUSIONS

The current AWWA D100 (and similar) codes employ seismic demand formulae which incorporate "R" response modification factors. These R factors appear to be historically based on similar factors for ductile building structures, and are NOT based on test data or empirical data.

The test and empirical evidence shows that the R values in the modern AWWA codes are not justified. The AWWA code does not disallow non-ductile detailing for attached pipes, and arguably allows bottom entry pipes to be located too close to side walls, especially for unanchored tanks.

The evidence suggests that the main reason that most steel tanks have not failed in past earthquakes is simply that the level of shaking was too low to result in wall uplift, coupled with the fact that the true buckling stress of the steel shell is substantially higher than code allowables. The good performance of many tanks cannot be attributed to ductile yielding of the steel shell.

The recommended changes to the AWWA (and similar) codes are as follows:

- If a working stress approach is retained, then limit R to be no higher than the true buckling stress divided by the code allowable buckling stress, limited to R = 2.5 (working stress approach). For commercial nuclear power plants, R = 1.
- Unanchored steel tanks may be used when the vertical wall compressive stress due to seismic overturning moment is computed considering cantilever, lift off and hoop breathing modes, and in consideration of whether or not a concrete ring beam or compliant foundation is used under the steel shell.
- Only anchored tanks should be used for important water storage tanks (I > 1).
- The use of Importance factors of I greater than 1 should be used to increase reliability for important tanks. Wherever post-earthquake performance is deemed important, the target PGA value should be the 475-year return period but no less than PGA = 0.20g. For pressure zones with two or more seismically-designed tanks (or one tank and one reliable pumped source), use I = 1.0 (1.25 in high fire threat areas). For pressure zones with only one tank, use I = 1.25 (1.5 in high fire threat areas).
- All tanks (whether anchored or not anchored) should be designed to accommodate wall uplift whenever the design is based on R > 1. The target uplift amounts should be at least 4 inches (anchored tanks) or 12 inches (unanchored tanks) for design of flexible connections of attached side-entry pipes. Any exterior attachments (power cables, etc.) should be similarly designed.
- No bottom entry pipes into the floor of a tank should be allowed within 24 inches (clear distance) of an anchored tank shell, or 48 inches of an unanchored tank shell whenever the design is based on R > 1.
- When using working stress design, the seismic allowable stresses under seismic loading shall be AWWA D100 values, including the effect of internal water pressure, whether anchored or unanchored. When using ultimate strength

methods with R = 1, the limiting buckling stress can be based on elastic-plastic considerations, maintaining a factor of safety of 1.5.

- If unanchored tanks are used, then all roof level support beams shall be designed to safely accommodate at least 4 inches of wall uplift, while maintaining vertical load carrying capacity of at least 20 psf. The roof should be checked for loads from sloshing, but in most cases damage to the roof will be due to wall uplift unless the roof is designed to accommodate such uplift.
- When selecting the grade of steel for tanks, a low yield stress steel will provide superior resistance to buckling. Using a high-yield stress steel will result in thinner walls, and less buckling resistance. If a high-yield stress steel is used, then careful attention should be made to assure that wall buckling is avoided using elastically-computed stresses.

UNITS

1 g = 386.4 inches / sec^2 = 9.81 meters /sec^2. 1 inch = 25.4 mm. 1 feet = 12 inches = 0.3048 meter. 1 psi = 1 pound per square inch = 6.89 kiloPascal = 6.89 kN/mm^2. 1 pcf = 1 pound per cubic foot. 1 ksi = 1,000 psi = 6,890 kN / mm^2. 1 MG = 1 million gallons = 3,785,413 liters. 1 pound = 1 pound force = 4.448 newtons.

REFERENCES

[1] AWWA Standard for Welded Steel Tanks for Water Storage, ANSI / AWWA D100-2005, -1996, -1984, -1978.

[2] Housner, G.W., Earthquake Pressures on Fluid Containers, Caltech, 1954.

[3] Dynamic Pressure on Fluid Containers, in Nuclear Reactors and Earthquakes, Technical Information Document 7024, Chapter 6 and Appendix F, prepared by Lockheed Aircraft Corporation, 1963.

[4] Wozniak, R.S. and Mitchell, W.W, Proposed Appendix P to API Standard 650, seismic Design of Storage Tanks, Basis of Seismic Design Provisions for Welded Steel Oil Storage Tanks, API Convention, Toronto, May, 1978.

[5] Niwa., A., Seismic Behavior of Tall Liquid Storage Tanks, EERC report 78/04, UC Berkeley, 1978.

[6]. Eidinger, J., Water System Fragilities, G&E Report R47.01.01 Rev 1, July 12, 2001, http://homepage.mac.com/eidinger/

Effects of Sloshing of Water in Receiving Water Tank on Water Distribution System during Earthquake

Masakatsu Miyajima and Kouichi Murata

ABSTRACT

The present paper focuses on an abrupt increase in flow rate and a decrease in water pressure of water distribution system in spite of no damage to pipeline during earthquake. First, an example of the 2004 Kii-hanto oki earthquake is given by using time histories of flow rate and water pressure recorded at Osaka City Waterworks Bureau (OWWB). The water distribution area of OWWB is divided into eighteen blocks and the water flow and water pressure of each block were recorded in each ten seconds after the 1995 Hyogoken nambu earthquake. The unusual phenomena of water distribution system, therefore, can be detected just after an earthquake. The occurrence of the unusual phenomena of water distribution system in the past earthquakes at OWWB after year of 2000 was shown and the relation to response velocity spectra of the past earthquakes were investigated. One of causes of the unusual phenomena is considered as sloshing of a water in receiving water tank. The dimensions of water receiving tanks in a block of Osaka city were investigated and the maximum displacement of water in the receiving water tank by sloshing was estimated.

Masakatsu Miyajima, Professor, School of Environmental Design, College of Science and Engineering, Kanazawa University , Kakuma-machi, Kanazawa, 920-1192 Japan

Kouichi Murata, Manager, Seibu Waterworks Maintenance Center, Osaka City Waterworks Bureau, 4-12-26 Minamihorie, Nishi-ku, Osaka 550-0015 Japan

INTRODUCTION

Unusual phenomena such as an abrupt increase in flow rate and a decrease in water pressure of water distribution system in spite of no damage to pipeline was sometimes occurred in the past earthquakes. In the 2008 Iwate miyagi nairiku earthquake, the unusual phenomena of water distribution system were occurred in Sendai City Waterworks Bureau. An abrupt increase in flow rate and a decrease in water pressure of water distribution system were occurred just after the earthquake and they continued about 10 minutes. Total flow rate of Sendai city increased about 45.5% of the average value and the water pressure decreased 28.6% of the average value at the maximum site. The similar phenomena also occurred in the water distribution system of the Niigata City Waterworks Bureau. These phenomena, however, did not affect life of citizen after the earthquake because of short duration.

Osaka City Waterworks Bureau (OWWB) sometimes recorded the phenomenon in the past earthquakes. The water distribution area of OWWB is divided into eighteen blocks and the water flow and water pressure of each block were recorded in each ten seconds after the 1995 Hyogoken nambu earthquake. The unusual phenomena of water distribution system, therefore, can be detected just after an earthquake.

The present paper focuses on an abrupt increase in flow rate and a decrease in water pressure of water distribution system in spite of no damage to pipeline during earthquake. First, an example of the 2004 Kii-hanto oki earthquake is given by using time histories of flow rate and water pressure recorded at OWWB. The occurrence of the unusual phenomena of water distribution system in the past earthquakes at OWWB after year of 2000 was shown and the relation to response velocity spectra of the past earthquakes were investigated. One of causes of the unusual phenomena is considered as sloshing of a water in receiving water tank. The dimensions of water receiving tanks in a block of Osaka city were investigated and the maximum displacement of water in the receiving water tank by sloshing was estimated.

UNUSUAL PHENOMENA OF WATER DISTRIBUTION SYSTEM IN OSAKA CITY WATERWORKS BUREAU DURING EARTHQUAKE

Unusual phenomena such as an abrupt increase in flow rate and a decrease in water pressure of water distribution system in spite of no damage to pipeline was occurred in OWWB in the past earthquakes. An example of the 2004 Kii-hanto oki earthquake is introduced here. Figure 1 shows time histories of the flow rate at the water distribution plants in Osaka City. The flow rate increased rapidly just after the earthquake. Total flow rate of Osaka city increased from about 73,000m^3/h to 101,000m^3/h (about 40%) just after the earthquake and higher flow rate continued during about 30 minutes. Figure 2 illustrates time histories of the water pressure at the water distribution plants. The lower water pressure continued during 3 to 10 minutes. Figure 3 shows the distribution of water pressure of each block just after the earthquake. The water pressure was more than 0.3MPa before the earthquake, but the water pressure decreased to less than 0.1MPa at seaside and highland areas just after the earthquake.

Table 1 lists the occurrence of the unusual phenomena of water distribution system in the past earthquakes at OWWB after year of 2000. According to Table 1, the unusual phenomena did not occur in the Kyoto-fu nambu earthquake in spite of three of the JMA (Japan Meteorological Agency) seismic intensity and the unusual phenomena occurred in the Geiyo earthquake of JMA seismic

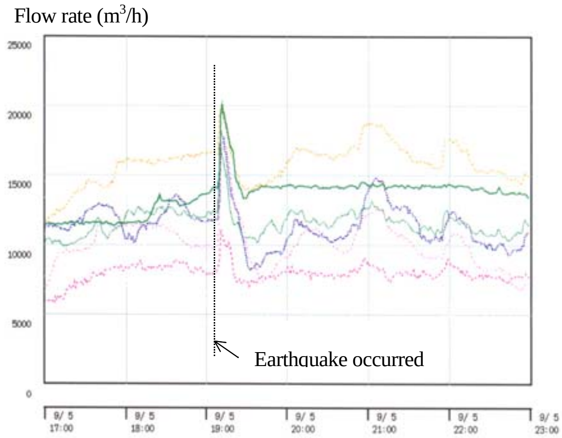


Figure 1. Time histories of the flow rate at the water distribution plants in Osaka City during the 2004 Kii-hanto oki earthquake.

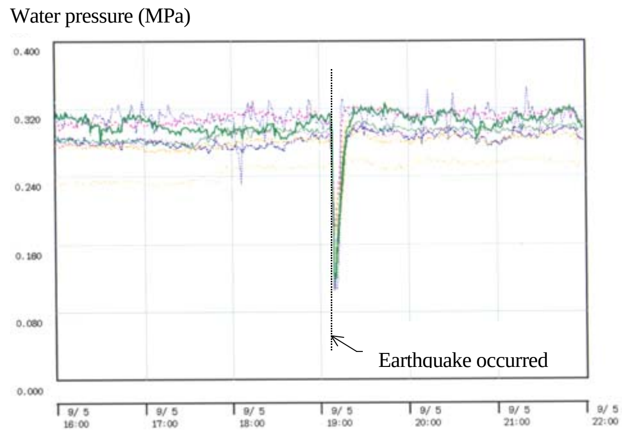


Figure 2. Time histories of the water pressure at the water distribution plants in Osaka City during the 2004 Kii-hanto oki earthquake.

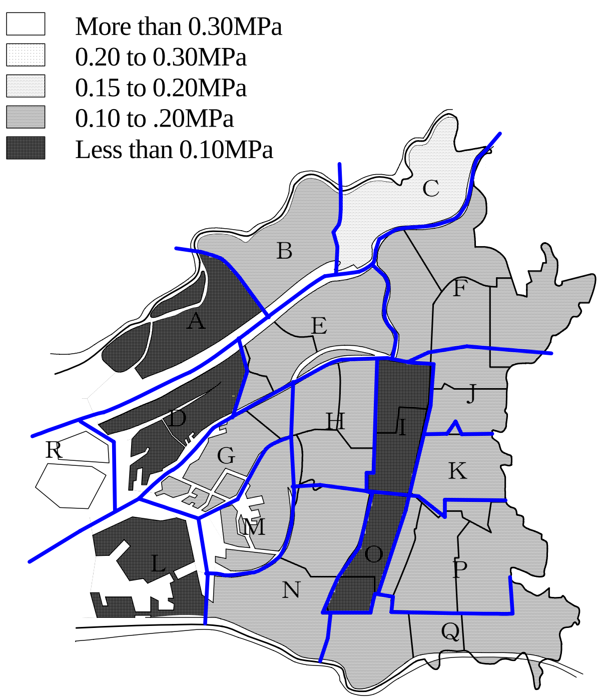


Figure 3. Distribution of water pressure of each block in Osaka city just after the 2004 Kii-hanto oki earthquake.

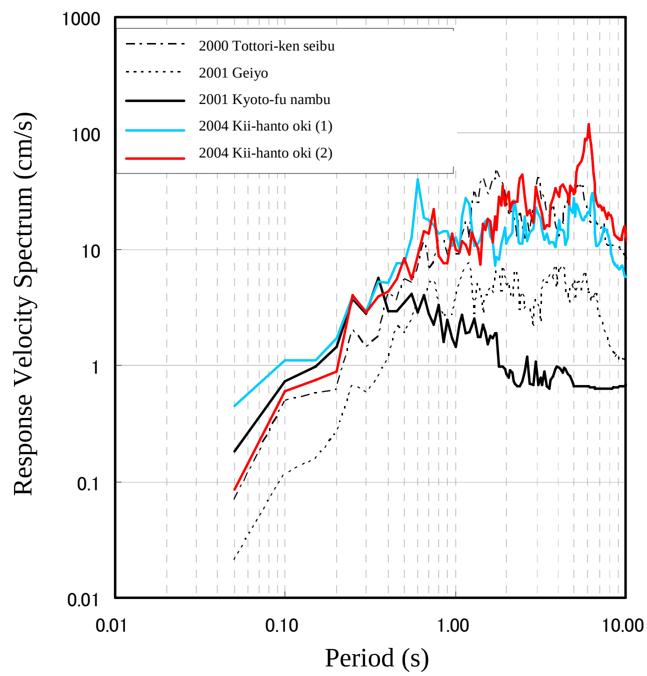


Figure 4. Response velocity spectra of recorded waveforms at Ooyodo water distribution plant of OWWB during the earthquakes.

nintensity two. This suggests that the unusual phenomena depends not only on the magnitude of ground shaking but also other factors such as frequency characteristics of ground shaking.

Figure 4 illustrates response velocity spectra of recorded waveforms at Ooyodo water distribution plant of OWWB during the earthquakes listed in Table 1. According to Figure 4, predominant periods of the Tottori-ken seibu, Geiyo, Kii-hanto oki earthquakes are more than one second, but that

TABLE 1. Occurrence of the unusual phenomena of water distribution system in the past earthquakes at OWWB after year of 2000.

Name of earthquake	Date	Time	Magunitude	SI in Osaka	Occurrence
Tottori-ken seibu	2000.10.6	13:30	7.3	4	Yes
Geiyo	2001.3.24	15:28	6.7	2	Yes
Kyoto-fu Nambu	2001.8.25	22:21	5.3	3	No
Kii-hanto oki (1)	2004.9.5	19:07	6.9	4	Yes
Kii-hanto oki (2)	2004.9.5	23:57	7.4	4	Yes

* SI : JMA Seismic Intensi

of the Kyoto-fu nambu earthquake is less than one second. The unusual phenomena occurred in the first four earthquakes and did not occur in the Kyoto-fu nambu earthquake. This suggests that the long period ground more than one second motion affects the unusual phenomena of the water distribution system.

ANALYSIS ON CAUSES OF UNUSUAL PHENOMENA

An analysis was conducted in order to clarify causes of the unusual phenomena of water distribution system after an earthquake. Increased water volume *Q* just after an earthquake is defined as Equation 1.

$$Q = \int_{t_0}^{t_1} q(t)dt - \frac{q(t_0) + q(t_1)}{2}(t_1 - t_0) \tag{1}$$

where $q(t)$: flow rate at time of t (m^3/h), t_0 : beginning time of unusual phenomena, t_1 : end of unusual phenomena

Figure 5 illustrates the relation between number of household per 1 ha and increased water volume in each block of Osaka city. If the abrupt increase in flow rate was caused by same behavior of citizens such as flush of toilet, the number of households has correlation with the increase of water volume. The regression line and correlation coefficient in each earthquake are also shown in Figure 5. Increased water volume is proportional to the number of households but the correlation coefficients are very low such as 0.1 to 0.3.

If sloshing of water in receiving water tank is occurred by an earthquake, draw of water to receiving water tank starts by error of sensor of water level in the receiving water tank as shown in Figure 6. Since this may be one of causes of unusual phenomena, the relation between increased water volume and number of receiving water tank per 1 ha in each block of Osaka city is shown in Figure 7. The correlation coefficients in Figure 7 are high. Sloshing of water in receiving water tank, therefore, may be one of the causes of unusual phenomena.

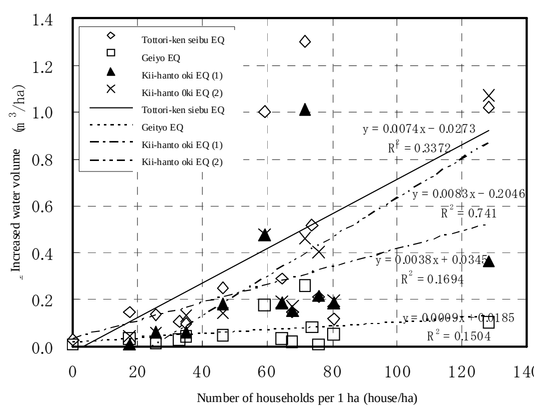


Figure 5. Relation between number of households and increased water volume in each block of Osaka city.

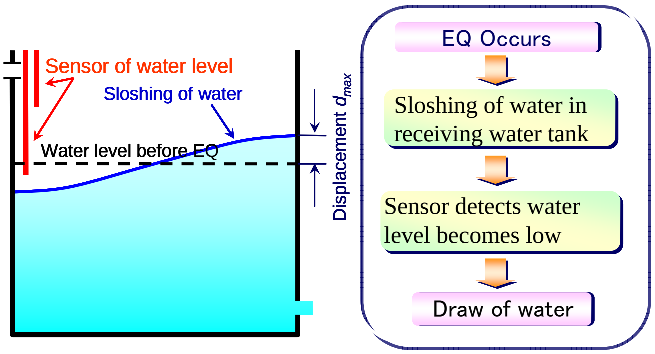


Figure 6. Mechanism of draw of water from distribution pipe to receiving tank by sloshing during earthquake.

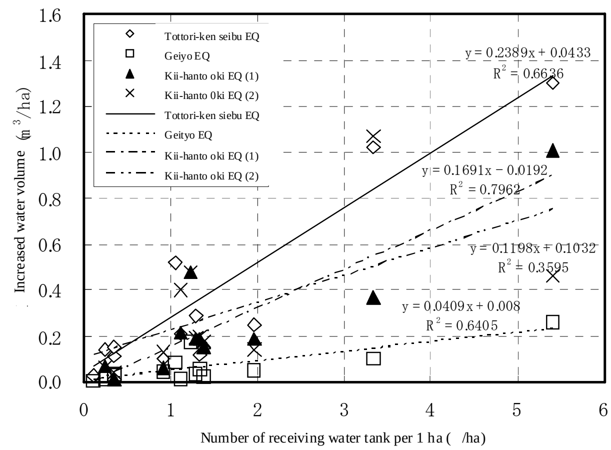


Figure 7. Relation between number of receiving water tanks and increased water volume in each block of Osaka city.

SLOSHING OF WATER IN RECEIVING WATER TANK

Occurrence of sloshing of water in receiving water tank depends on the dimensions of receiving water tank and the height of water in the tank. Dimensions of receiving water tank were investigated in Sakishima block (See Figure 8) of Osaka city. There are 121 receiving water tanks, but its dimensions are known for only 63 receiving water tanks in Sakishima block according to the data base of OWWB. Figure 9 illustrates the relation between capacity of water receiving water tank and its height. The height of the tank consists between 1m to 4m in spite of the capacity. The height of the tank depends on the height of floor of a building.

Sloshing of water in receiving water tank occurs when a predominant period of ground shaking coincides with a natural period of sloshing of water in receiving water tank. A natural period of sloshing is given by Equation 2.

$$Ts = \frac{2\pi}{\sqrt{1.58\frac{g}{l}\tanh\left(1.58\frac{h}{l}\right)}} \tag{2}$$

where,

Ts : Predominant period of sloshing (s)
g : Acceleration of gravity (m/s^2)
h : Depth of water (m)
l : 1/2 of length of basement (m)

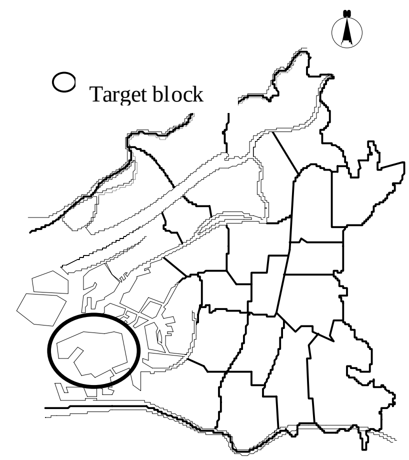


Figure 8. Location of target block, Sakishima area.

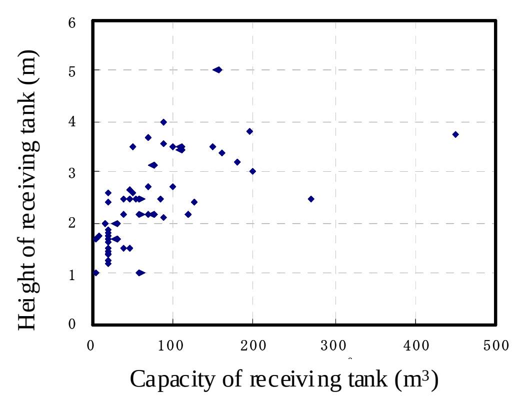


Figure 9. Relation between capacity of water receiving water tank and its height.

This equation indicates that the natural period of sloshing depends on a height of water and length of base of the tank. The natural periods of sloshing of water in receiving water tanks in Sakishima block were calculated by using Equation 2. Figure 10 shows cumulative percentage of natural period of water in receiving water tank in case that the height of water is 3/4 and 1/2 of the height of water tank. The height of water is variable and depends on use of water. The natural period of sloshing is more than 1.0 second of more than 80% of the water tank in the direction of long side in Sakishima block

The maximum displacement of water caused by sloshing was estimated by using Equation 3. A response velocity spectrum of earthquake waveform is used in this equation.

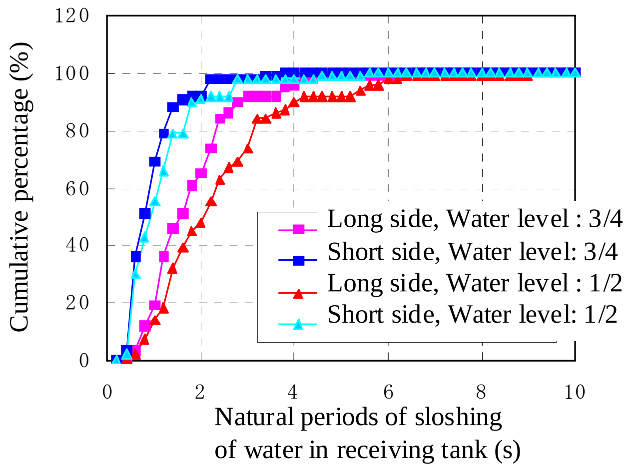


Figure 10. Cumulative percentage of natural period of water in receiving water tank

$$d_{\max} = \frac{0.527 l \coth\left(1.58\frac{h}{l}\right)}{\dfrac{g}{\omega^2 \theta_h l} - 1} \qquad (3)$$

$$\theta_h = 1.58 \frac{S_v}{\omega l} \tanh\left(1.58\frac{h}{l}\right) \qquad (4)$$

where,

g : Acceleration of gravity (m/s^2)
h : Depth of water (m)
l : 1/2 of length of basement (m)
S_v: Response velocity spectrum of predominant period of sloshing (m/s)
ω :Predominant circular frequency

By using waveforms of earthquakes listed in Table 1, the maximum displacement of water was calculated. Figure 11 illustrates cumulative percentage of the maximum displacement of water caused by sloshing in each earthquake waveform listed in Table 1. The height of water in receiving water tank is assumed as 1/2 and 3/4 of the height of water tank. Both directions of long and short side of base were calculated. The maximum displacement of water was more than 0.1m for more than half of all water tanks in Sakishima block in case of 1/2 water level of height of water tank during Tottori-ken seibu and Kii-hanto oki earthquakes. On the other hand, the maximum displacement of water was less than 0.05m for more than 90% of all water tanks in Sakishima block. This suggests that the sloshing of water in receiving water tank is one of causes of unusual phenomena of the water distribution system after an earthquake.

Finally, the maximum displacement of water caused by sloshing was estimated by using a scenario earthquake waveform of Tonankai-nankai earthquake. Figure 12 illustrates response velocity spectrum of a scenario earthquake waveform of Tonankai-nankai earthquake. Long period more than 1.0 second is predominant very much. Figure 13 indicates the maximum displacement of water in the direction of long side in case of 1/2 water level of height of water tank. According to this

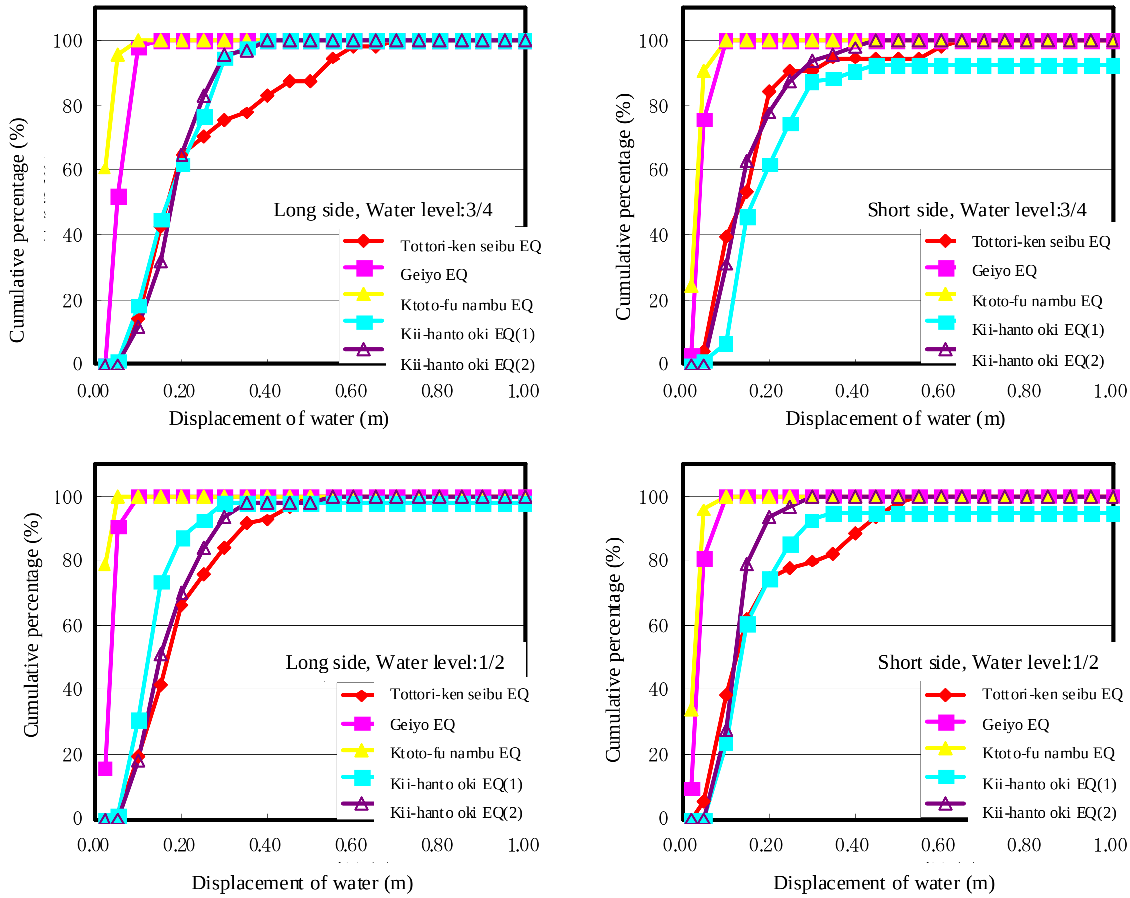


Figure 11. Cumulative percentage of the maximum displacement of water caused by sloshing in each earthquake waveform listed in Table 1.

figure, the maximum displacement of water was more than 0.5m for more than 60% of all water tanks in Sakishima block. This value is much greater than that of Tottori-ken seibu earthquake. These phenomena induced by sloshing may affect life of citizen directly after the earthquake

CONCLUSIONS

This study deals with an abrupt increase in flow rate and a decrease in water pressure of water distribution system in spite of no damage to pipeline during earthquake. The conclusions drawn from this study are summarized below.

1. Occurrence of the unusual phenomena of water distribution system in the past earthquakes at OWWB was investigated. Although an unusual phenomena did not occur in the Kyoto south earthquake of JMA seismic intensity three, the unusual phenomena occurred in the Geiyo earthquake of JMA seismic intensity two. This suggests that the long period ground motion affects the unusual phenomena of the water distribution system.

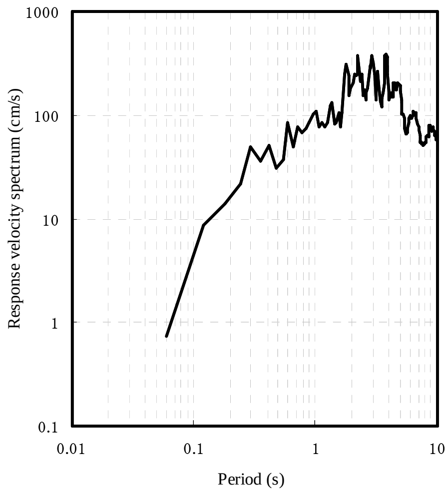


Figure 12. Response velocity spectrum of a scenario earthquake waveform of Tonankai-nankai earthquake.

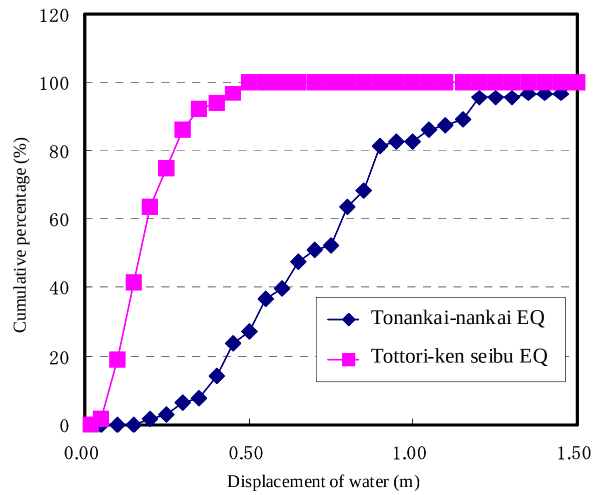


Figure 13. Maximum displacement of water in the direction of long side in case of 1/2 water level of height of water tank.

2. The correlation between the number of receiving water tank and increased water volume was high. If sloshing of water in receiving water tank is occurred by an earthquake, draw of water to receiving water tank from pipeline starts by error of sensor of water level in the receiving water tank. Sloshing of water in receiving water tank, therefore, may be one of the causes of unusual phenomena.
3. The maximum displacement of water caused by sloshing was estimated by using a scenario earthquake waveform of Tonankai-nankai earthquake. The maximum displacement of water was more than 0.5m for more than 60% of all water tanks in this example. Tshi value is much greater than that in the Tottori-ken seibu earthquake. These phenomena induced by sloshing may affect life of citizen severely after the earthquake.

ACKNOWLEDGEMENTS

This study was supported in part by the Grant-in-Aid for Scientific Research from the Ministry of Education, Culture, Sports, Science and Technology, Japan (No. 20310108).

REFERENCES

[1] Murata, K. and Miyajima, M.: Influence of Receiving Water Tank Sloshing on Water Distribution System, Journal of Japan Earthquake Engineering, Vol. 7, No. 1, pp. 27-42, 2007 (in Japanese).

[2] Japan Waterworks Association: Seismic Design and Construction Guidelines for Water Supply Facilities 2009 (in Japanese).

From Chi-Chi Earthquake to Study about Aseismic Measures and Disaster Crisis Management in Taipei

Chin-Tse Cheng[1]* and Jiin-Shyang Chen[2]

ABSTRACT

Taiwan is located in a complex, tectonically active region. This reflects have high tectonic stresses in the Taiwan collision zoned that give rise to the large number of damage earthquakes like as Chichi earthquake. The production water of Taipei Water Department (TWD) is life consumption water for residents in Great Taipei Area in Northern part of Taiwan. The total area of supply is 434 square KM with total population of about 3.85 million users. From the consideration of probabilistic models about seismic analysis research, the results provide a crisis potential analysis to identify the risk distribution in Great Taipei Area.

The countermeasures and strategies of emergency response will be designed to make fundamental suggestions toward pipeline system under major earthquake attack. The results of study are as follows: 1.The major pipeline bridge has enough aseismic capacity to resist the earthquake (PGA is 0.23g) and provide the main pipeline. 2. According to the underground embedment ductility iron pipeline analysis result, discovered its main destruction form is by attaches the bend distortion destruction primarily, destruction scope in 2 sections of tubes lengths. 3. According to PCCP mechanics analysis result, discovered reaches above 75 cm when the surface relative displacement, can have the connector destruction and to create the water distribution function to lose. 4. According to potential analysis results, some districts & cities are potential higher regions.

From water utility management and crisis response & management that reserves capacity is necessary. Then, expect to lower the water supply risk and achieve stable supplying by organizing backup systems of water resources and supplies in Taipei. TWD has planed expansion project like the 5th phase of Taipei Water Supply Expansion Plan that have constructing a safer and steadier water system from yr. 1991, 2030 as the goal year. In the other hand TWD had formulated improvement plan and has conducted for example Taipei City Water Supply Pipe Net-work Improvement Plan from yr. 2006 to 2025 that had been formulated are improvement on: water mains and primary distribution pipelines; emergency water supply facilities; booster station; Geographic Information System (GIS) and water distribution Supervisory control and data acquisition (SCADA) system; and guidelines for design and construction, etc.

In general, no plan can totally prevent the impact of seismic. In order to cope with earthquakes and other natural disasters, the TWD has developed a range of emergency crisis management procedures that utilize existing water distribution ponds and pumping stations, with a total of 30 emergency water distribution stations that can provide up to 25 days of emergency life water for residents of the area and give workers enough time to repair any facilities damaged by the disaster.

Keywords: Aseismatic Measures, Disaster Prevention, Crisis Management, Emergency Life-Water

---

[1]*Chin-Tse Cheng, Senior Engineer& Manager, Taipei Water Department, 131, Changsing St., Taipei 10672, Taiwan, ROC.

[2]Jiin-Shyang Chen, Chief Engineer, Taipei Water Department, 131, Changsing St., Taipei 10672, Taiwan, ROC,

INTRODUCTION

Taiwan is in a region with frequent earthquakes at the west Pacific Ocean. Philippine Sea Plate is subducting northwestward along the Ryukyu Trench in the north. Eurasian Plate underthrusts the Philippine Sea Plate along the Manila Trench in the south. The Philippine Sea Plate is moving northwestward at the rate of 7 cm/yr relative to the Eurasian Plate, creating the Taiwan collision zone. This reflects have high tectonic stresses in the Taiwan collision zone that give rise to the large number of damage earthquakes. Several devastating earthquakes occurred in the last century and caused significant damage to structures and economical losses. The latest one was the Chi-Chi earthquake occurred on September 21, 1999 and caused by thrust-slip fault movement. It resulted in 2,456 deaths, 10,718 injured, 53,661 houses fully destroyed, and 53,024 houses damaged.

There were severe water supply systems destructions in Taiwan, such as intake dam, steel trunk and distribution reservoir, pipelines, etc. Indeed, as a public utility operator under the Taipei Municipal Government's jurisdiction, TWD is responsible for supplying some 2.5 million cubic meters of high-quality potable water a day to over 3.85 million users in the Taipei metropolis. Tap water supply system containing huge and complicated underground pipelines is the fundamental infrastructure of urban city. For increasing the post-quake survival and function of modern city, evaluating and strengthening of the seismic capacity in existing water supply system will contribute to improving daily safety [1]. Therefore we need to study about aseismic measures and disaster crisis management in Taipei metropolitan water lifeline system.

ASEISMATIC MEASURES

How to effort to mitigate, prevent and response disasters that need to know the problems. Most citizens of Taiwan intend to have a fatalistic attitude and a negatives standpoint to hazards. The local governments and communities disaster management capability is insufficient. Make more efforts at emergency response than hazard mitigation. Most strategies of hazard mitigation plans are not specific. Due to need high budget, so that do not take full advantage of research results. Disaster prevention and response is one of the most important government tasks after 921 earthquake. Since the 'Disaster Prevention and Response Act' (DPRA) was promulgated in yr.2000. Disaster prevention and rescue has transformed from traditional two-dimensional operations to a cross-department integrate collaborative task. Three-level disaster management system, Central, County/City, Township, is established. Each level government needs to develop" Disaster Prevention and Response Plan(DPRP).

To explore the seismic behavior of pipelines, TWD had several evaluations about seismic disaster, in which lately one project [2] conducts the numerical simulation by adopting finite-element software, ANSYS and SAP2000 [3], to identify the seismic insufficiency. The main analysis parameters include pipe diameters, section length, connector types and pipe materials, which will be applied to evaluate the location and severity in the most risk potential areas (shown in Fig.1 & Fig.2). In addition to behavior analysis of underground pipeline, the pipeline bridge is one of research topics in the report to explore the dynamic response of structural system.

To have practical applications of research result, the seismic evaluation of pipelines according to PGA, PGD and repair rate will provide the basic estimation of manpower and time for post-disaster restoration. In process of study, the disastrous areas in central Taiwan are selected as the regression datasets to formulate the regression equations for damage estimation. With the further consideration of probabilistic models to include risk analysis and scenario simulation, the study result will provide a risk potential analysis to identify the risk distribution in metropolitan Taipei. The countermeasures and strategies of emergency response will be designed to make fundamental suggestions toward pipeline system under major earthquake attack. The results of the present study are as follows:

1) The pipeline bridge has enough aseismic capacity to resist the earthquake (PGA is 0.23g) and provide the main pipeline.

2) According to the underground embedment ductility iron pipeline analysis result, discovered its main destruction form is by attaches the bend distortion destruction primarily, destruction scope in 2 sections of tubes lengths.

3) According to PCCP mechanics analysis result, discovered reaches above 75 cm when the surface relative displacement, can have the connector destruction and to create the water distribution function to lose.

4) The present study develops the new formula which is more accurate than foreign experiential formula to estimation the repair rate of pipeline.

5) According to potential analysis results, Shilin district, Daan district, Sindian city and Jhonghe city are potential higher regions.

Figure 1. Diagram of seismatic soil liquefaction potentiality in Taipei Supply Area

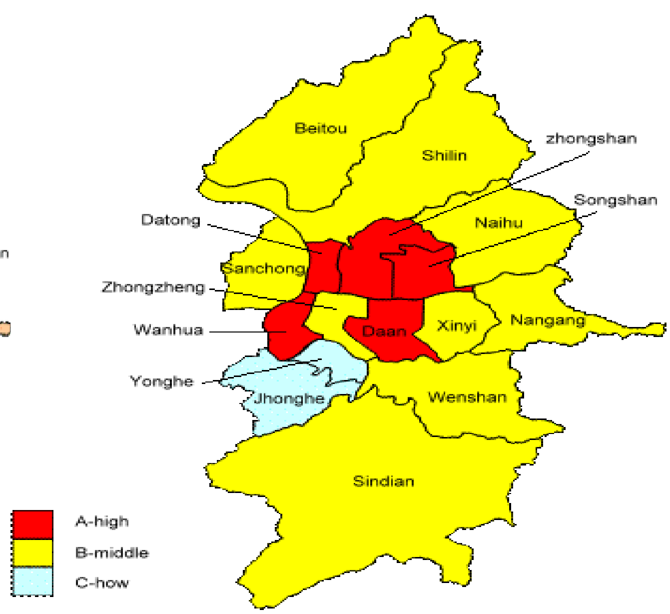


Figure 2. Diagram of seismatic surface displacement potentiality in Taipei Supply Area

The main suggestions of the present study are as follows:

1) Dives A, B, the C three potential regions according to the surface displacement, and each region proposed the design proposal.

2) The research provide the American ALA codes and design proposal of attachments and the pipeline pass the fault to be referred.

3) Suggest to set disaster scale to prepare according to suitably return period (for example 500 years).

4) Suggest to plan emergency water supply place according to this research institute estimate.

DISASTER CRISIS MANAGEMENT

It is getting more difficult day by day in developing water resources in Taiwan area and climate change has affected both the supply and demand of water. Resulted from global weather change and warming, too much and too less rainfall (flood and drought) will occur very often and TWD shall be prepared for them（shown in Fig.3 & Fig.4）. The droughts in northern Taiwan in 2002 and 2003 caused serious impact on urban social activities. Typhoon is another serious factor. Example is Typhoon Eire of recent years that caused extensive water outage in Taoyuan area. 1999 Chi-Chi Devastating earthquake damaged part of water supply facility and affect water supply. With some previous lessons, water utilities shall upgrade emergency response ability and cooperate with other water resources department to maintain the operation and management with sufficient surplus to ensure satisfaction of customers.

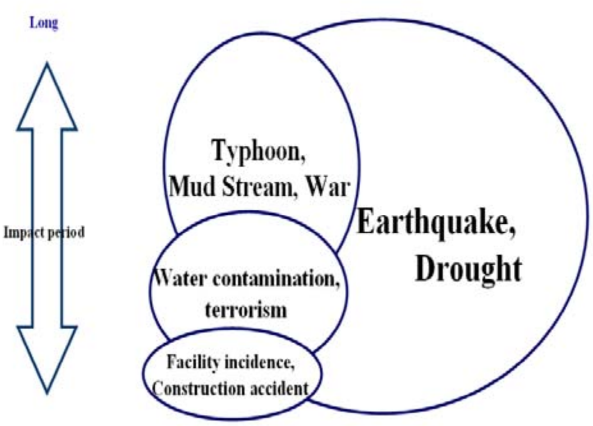
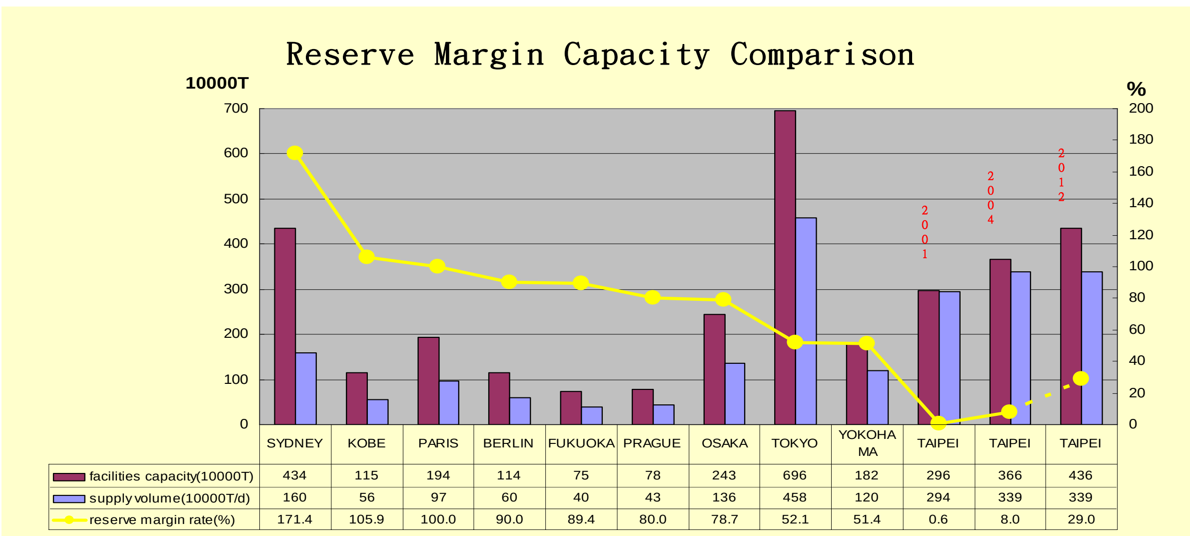


Figure 3. Diagram of Impact on Water Supply Resulted From Hazards

Figure 4. TWD Water Supply Hazard Preventing

Tap-water supply is extremely important to resident and users in the service area, it is therefore necessary for water utilities to make timely review of weakness to be ready for risk and to avoid occurrence of crisis, and ready them to offer stable, safe and quality water and to meet with the complicate needs and trend of rising cost. In this aspect, water utilities shall take effective, understandable planning and construction for customers [4]. The main part of countermeasure in dealing with earthquake hazard is the strengthening of facility system. The higher the safety of facility system is, the risk of being damage will be lower. The facility system strengthening goals of TWD include: Taipei water supply expansion 5th phase plan [5] and water supply pipe net-work improvement plan [6] etc.

Although Taipei district reservoir storage capacity is large, but from water utility management and crisis response & management that reserves capacity is necessary. In the 16 modern cities of globe that purification plants capacity reserve margin is from 51% to 171%. TWD has reserve Margin about 20% original, but it need to support emerge water temporary to other place like as Ban-Xin to Tao-Yuan or Kee-lung metropolitan, so that total reserve margin in the northern area of Taiwan is very low（shown in Fig.5 & Fig.6）. It was happen several deficient water events due to typhoon or other disaster in recently several years [7]. Therefore, we need actively to plan expansibility to meet the forecasted water demand include reserve margin capacity for disaster or maintenance or rehabilitation or emerge water support temporary [8].



	SYDNEY	KOBE	PARIS	BERLIN	FUKUOKA	PRAGUE	OSAKA	TOKYO	YOKOHAMA	TAIPEI	TAIPEI	TAIPEI
facilities capacity(10000T)	434	115	194	114	75	78	243	696	182	296	366	436
supply volume(10000T/d)	160	56	97	60	40	43	136	458	120	294	339	339
reserve margin rate(%)	171.4	105.9	100.0	90.0	89.4	80.0	78.7	52.1	51.4	0.6	8.0	29.0

Figure 5. Diagram of Taipei Water Supply reserve margin capacity comparison

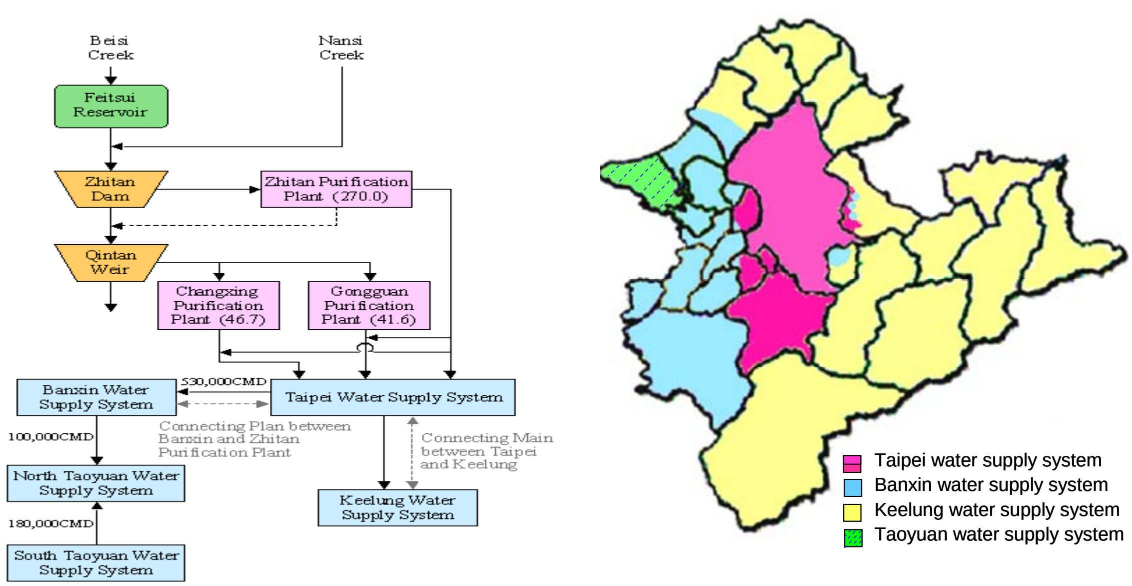


Figure 6. Diagram of Taipei Emerge Temporary Water Supply

A. Taipei Water Supply Expansion 5th Phase Plan

In order to support local sub-demand within the system each other. Then, expected to lower the water supply risk and achieve stable supplying by organizing backup systems of water resources and supplies in Taipei. TWD has planed expansion project like the 5th phase of Taipei Water Supply Expansion Plan that have constructing a safer and steadier water system from 1991, 2030 as the goal year (37.2billions NT dollars). The plan also could be prevent deficient water crisis because it enhance capacity reserve Margin. For example the No. 5 purification plant at Chihtan with a daily processing capacity of 700,000 cubic meters finished in 2004（Chihtan purification plant total capacity is 2.7 million cubic meters）. Except building purification plants to match water demand and spare capacity, constructing a strong dual water supply system for each water supply region is the principal work. On the purified water , the first line, 3400mm~2000mm in diameter and 17.3Km in length and completed in 1984, the second line, 3800mm~2400mm in diameter and 17.4Km in length and completed in 2000, connect 9 main boosting station transport purified water by gravity from the Chihtan Water Purification Plant. To each water supply region, the water can come from two or more boosting pump station through the trunk line of its distribution system to satisfy the water demand. At the same if the mishap is occurred on any part of huge trunk system or any boosting station or any part of the trunk distribution system, the water can be regulated to support each other during doing rush repairs or crisis management（shown in Fig.7）.

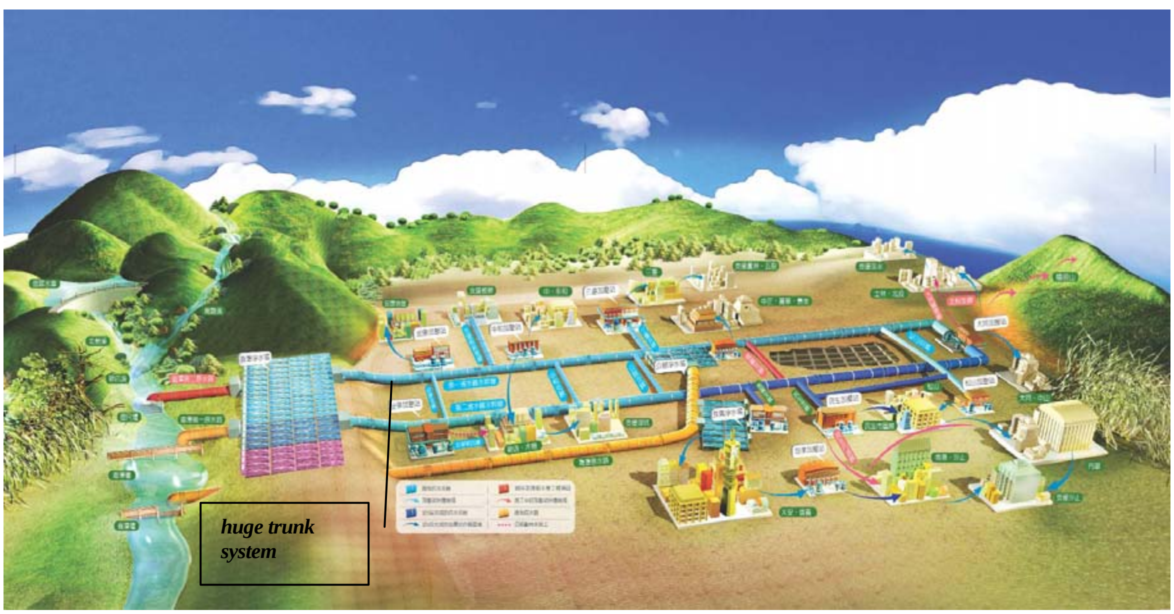


Figure 7. Diagram of Taipei Water Supply Expansion 5th Phase Plan- backup systems

B. Water Supply Pipe Net-work Improvement Plan

TWD had formulated improvement plan and has conducted for example Taipei City Water Supply Pipe Net-work Improvement Plan (20 billions NT dollars) that had been formulated are improvement on: water mains and primary distribution pipelines（shown in Fig.8）; emergency water supply facilities; booster station; local redundant piping network; Geographic Information System

(GIS) and water distribution Supervisory control and data acquisition (SCADA) system; and guidelines for design and construction, etc. It is anticipated that the above improvements will resolve many problems currently encountered by the TWD.

Water utilities have recognized the need for comprehensive analysis of their distribution systems in order to identify all pipes in need of renewal and improvement. GIS, hydraulic model, water quality model and other information management technologies have helped to collect maintain and analyse data on a system-wide basis. Basic factors as Age and material of the pipe, number and frequency of main breaks, reduction in hydraulic capacity, water quality problems, joint types and related joint damages, other strategic considerations, pavement overlay program, etc. We established a priority order for improvements based on such factors and formulated systematic and efficient plans for replacement and rehabilitation.

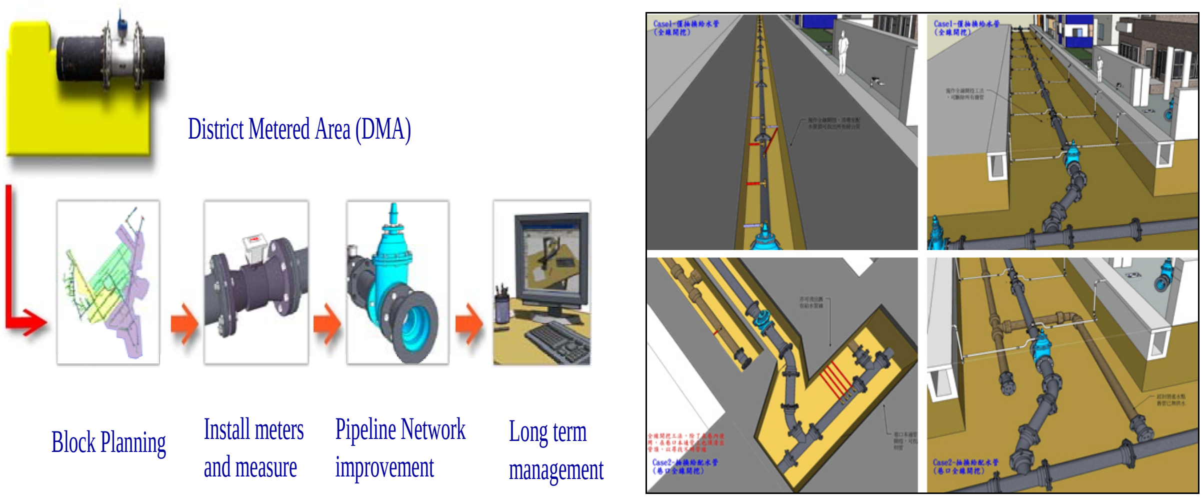


Figure 8. Diagram of Water Supply Pipe Net-work Improvement Plan-pipeline improvement

C. Emergency Water Supply Countermeasures Plan

After the devastating Chi-Chi earthquake, many of water supply system in central Taiwan was heavily damaged, the water department of Taipei city government austerely attends to the risk of water supply damages in great earthquakes. The water department has three anti-seismic measures nowadays: water supply system strengthening plans, emergency recovery measures, and emergency water supply measures [9]-[11]. The emergency water supply countermeasures include emergency water tanks, emergency underground water supply plans, water supply trucks, and so on (shown in Fig.9). We hope to continuously collect more excellent advice to improve the department's aseismic measures.

When a major hazard occurs on water supply equipment and mains, it will cause damage and interrupt normal supply of water for a short period of time, which will of course jeopardize the interest of general public in access to basic life support water. Since 2004, TWD has gradually installed emergency water supply stations in Taipei. Up to 2008, 30 facilities have been completed (shown in Fig.10 & Fig.11). They are supposed to provide 300,000 tons of life supporting drinking water to 3.87 million people in the service jurisdiction for 25 days. Subsequently, in coupling with the Hazard Preventing Park plan of Taipei City Government, 3 new water storage tanks of pipe shape (shown in Fig.12 & Fig.13) and 4 reinforced concrete storage tanks will be built in the following years. The design idea is to set emergency shutoff valves into the accessing pipes to storage tanks. At occurrence of the hazard, the emergency shutoff valve will be shut automatically and retain water in storage tank. Thus, people can take water from the storage facilities for life supporting purpose.

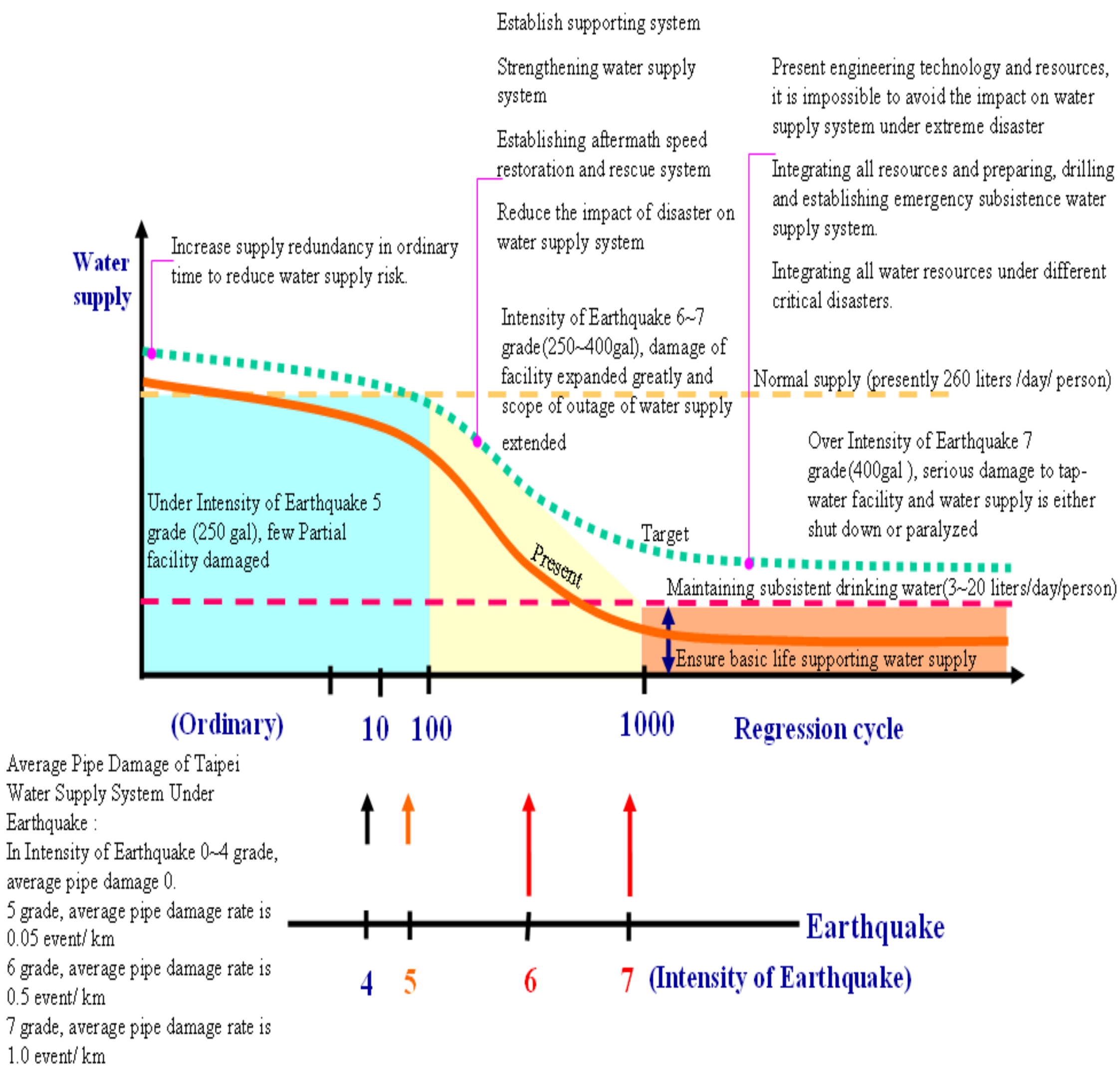


Figure 9. Diagram of Taipei Water Supply under the Regression Cycle of Earthquake

We wand to present the goal, content and execution realities of the Emergency Water Supply Station Establishing Plan of TWD and the establishing experience of similar facilities in other countries [12],[13]. We also present our life-supporting storage tank designs, emergency shutoff valves, flexible pipes, model test and the problems we encountered. Finally, the programs adopted to deal with the problems and the future developments are discussed as well.

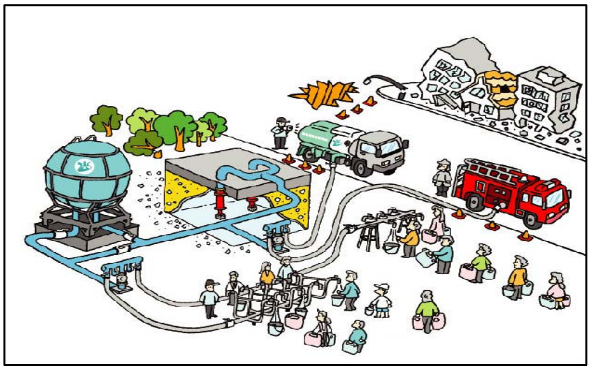


Figure10. Emergency Water Drawing from Distribution Pond

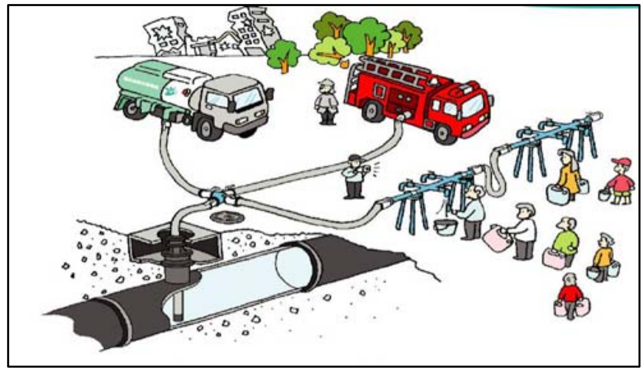


Figure11. Trunk Line Emergency Water Drawing Station

Figure 12. Site Construction(1/2)

Figure 13. Site Construction(2/2)

CONCLUSION

We all know, the devastating earthquake is not avoidable in Taiwan area, and in order to strengthen response capacity of tap-water system toward earthquake, in addition to establish complete hazard protection and minimization system in conjunction with hazard prevention and rescue units, water utilities shall build complete and proper water supply pipeline network, promote the adjustment and response ability in water supply , so as to control the scope of damage to keep it from expanding and / or deepening after occurrence of earthquake, or secondary hazard.

The Taipei area emergency water supply plan is planned basis the existing water supply system, and the emergency water drawing stations are located based on distribution ponds and available transporting trunk line or in shelter parks, they could not be located evenly and the density will not be sufficient. In future, the even distribution principle shall be attended by finding other feasible places for establishing emergency water supply stations, so that the system may be as perfect as possible.

Finally, TWD will still facing natural disaster, which may cause hazard to tap-water equipment, and in meeting with the global warming and abnormal rainfall, TWD needs to review water source and equipment from time to time to meet with demand of water supply. In the weakness of water supply flow, TWD needs to check for countermeasures from technical, institutional and decision aspects, and shall raise funds to improve aged and poor pipelines and equipment for stabilizing water supply.

REFERENCES

[1] Chin-Tse Cheng, 「A Study in the Aseismic Measures for Improvement of Taipei Water Supply System」, 12th IWA-ASPAC Regional Conference and Exhibition, 5-9 Nov. 2000 Chiangmai THAILAND。

[2] National Center for Research on Earthquake Engineering, 「Taipei Area underground pipelines seismic evaluation research」, Taipei Water Dept., 2006.

[3] Wilson, Edward L. etc.（1999）,「SAP2000 Analysis Reference」, Computers and Structures, Inc., Berkeley, California, U.S.A. 。

[4] Chin-Tse Cheng, "The Experience of Water Security in Taipei", IWA 5th World Water Congress and Exhibition, 2006, Beijing, P.O.C. 。

[5] Sinotech Engineering Consultants Ltd., Subsequent Plan for the Water Supply Project of 5th Construction of Taipei Area, Taipei Water Dept., June 2006.

[6]「Water Supply Pipe Net-work Improvement Plan」, Taipei Water Dept., 2004.

[7] Chin-Tse Cheng etc.,「A Study about New Crisis Management Model In Taipei Water System from Retrofit Chingtan Raw Water」, The First International Conference on Sustainable Water Environment: Water Resource and Quality Management, 2005, Taipei, R.O.C.。

[8] Chin-Tse Cheng, 「An experience in the benefit of the enhancement of reserve margin capacity in Taipei water supply system to the crisis management of water resource and utility」, IWA 5th World Water Congress and Exhibition 2006, Beijing, P.O.C. 。

[9] Sinotech Engineering Consultant Ltd., Report of Assessment Project for Earthquake Resisting Distribution Pond, Taipei Water Dept., 1998 。

[10] Chin-Tse Cheng etc., 「A Study of Taipei District Aseismic Deep Well & Water Tank Apply to Life Water」", Taipei City Government Research Plan Report, 2001。

[11]Cheng chin-tse, 「A Study of Taipei District Water Supply System Distribution Rationalization & Crisis Management」, Taipei City Government Research Plan Report, 2002。

[12]Report on Tour of Tokyo, Japan for Leak Improvement and Pipeline Replacement Plan, as well as Quake Resistant Emergency Water Supply System (2004) 、(2007) 、（2009）, Taipei Water Department, Taipei, R.O.C.

[13]Study on Establishing Emergency Water Supply Facilities to the Existing Water Supply System, Taipei Water Department (2007) Taipei, R.O.C. 。

Study for the Safety of Ductile Iron Pipe Earthquake-Resistant Joint in Massive Earthquake

Yoshinori Itani*1
Shogo Kaneko*1

ABSTRACT

Excellent anti-seismic performance of Ductile Iron Pipe Earthquake-Resistant Joint has been proven by the fact that up to now no damages have been found in the earthquakes occurred in Japan. In this paper, then, we examined the safety of the Ductile Iron Pipe Earthquake-Resistant Joint under extremely large ground strain generated by the ground shaking. And moreover, we confirmed the safety of the Ductile Iron Pipe Earthquake-Resistant Joint by the numerical analysis of buried pipeline assuming large ground displacement such as active faults.

1. INTRODUCTION

The 1995 Hyogoken-Nanbu Earthquake ("Kobe Earthquake") caused severe damages that had not been experienced in Japan since the end of World War II. Although the Kobe Earthquake also caused unprecedented damages on water pipelines, the Ductile Iron Pipe Earthquake-Resistant Joints ("ER -Joints") had no damages. The ER-Joints had no damages in a lot of earthquakes after the Kobe Earthquake, a fact that demonstrates their high resistance to earthquake. However, it is assumed that the massive earthquakes may occur due to the displacement of oceanic plates or active faults, and the possibility of the massive earthquakes that have never been experienced is incontrovertible.

In general, a buried water pipeline can be damaged by ground shaking or permanent ground displacement in earthquakes.

Regarding ground shaking, Japan Water Works Association analyzed the seismic waveforms recorded during the Kobe Earthquake to develop Seismic Design and Construction Guidelines for Water Supply Facilities [1]. The guidelines require that the maximum velocity response must be 100 cm/s for a seismic ground motion level of 2. Since this velocity response corresponds to a ground strain of less than 1.0%, the ER-Joints can completely absorb the ground strain caused by seismic ground motions.

On the other hand, regarding permanent ground displacement, massive liquefaction induced permanent ground displacement that caused devastating damage to the stricken area in the Kobe Earthquake. The Kobe Earthquake inspired many researchers to study liquefaction in a wide variety of aspects. For example, Miura[2] found that the ER-Joints buried on a reclaimed island fully exhibited their slip-out resistance by flexibly following a ground strain of 1.7%, even when the island was liquefied in the Kobe Earthquake.

In the present study, the authors sought to clarify the safety of the ER-Joints in seismic ground motion with a velocity response as unprecedentedly high as 300 cm/s. For ground displacement, we investigated the safety of the ER-Joints to displacement by an active fault. This paper reports the study results.

2. RECENT EARTHQUAKES AND THE ER-JOINTS

Areas hit by earthquakes with a seismic intensity of 6 or above during and after 1993 are shown in Figure 1. As this figure shows, earthquakes with a seismic intensity of 6 or above strike Japan at a frequency of 1.4 times per year. This indicates that Japan is now in a period of brisk seismic activity.

Bureaus of water-supply are actively promoting the seismic strengthening of water pipelines. The ER-Joints are usually used to improve the earthquake-resistance of water pipelines. The performance

*1 Kubota Corporation, Ductile Iron Pipe R&D Dept.

of NS type joint, one of the typical ER-Joints, is shown in Figure 2. These joints can expand, contract and deflect, and have slip-out resistance, sufficient to resist the external force exerted on them in an earthquake. This performance of the ER-Joints is categorized the highest class in ISO 16134. These joints installed in Japan have never been damaged by earthquakes.

An overall 1,300 km or more of ductile iron pipeline employing the ER-Joints is in operation in the areas that were hit by earthquakes of a seismic intensity of 6 or above. Figure 3 shows pictures of ductile iron pipes with the ER-Joints that withstood liquefaction and ground collapse.

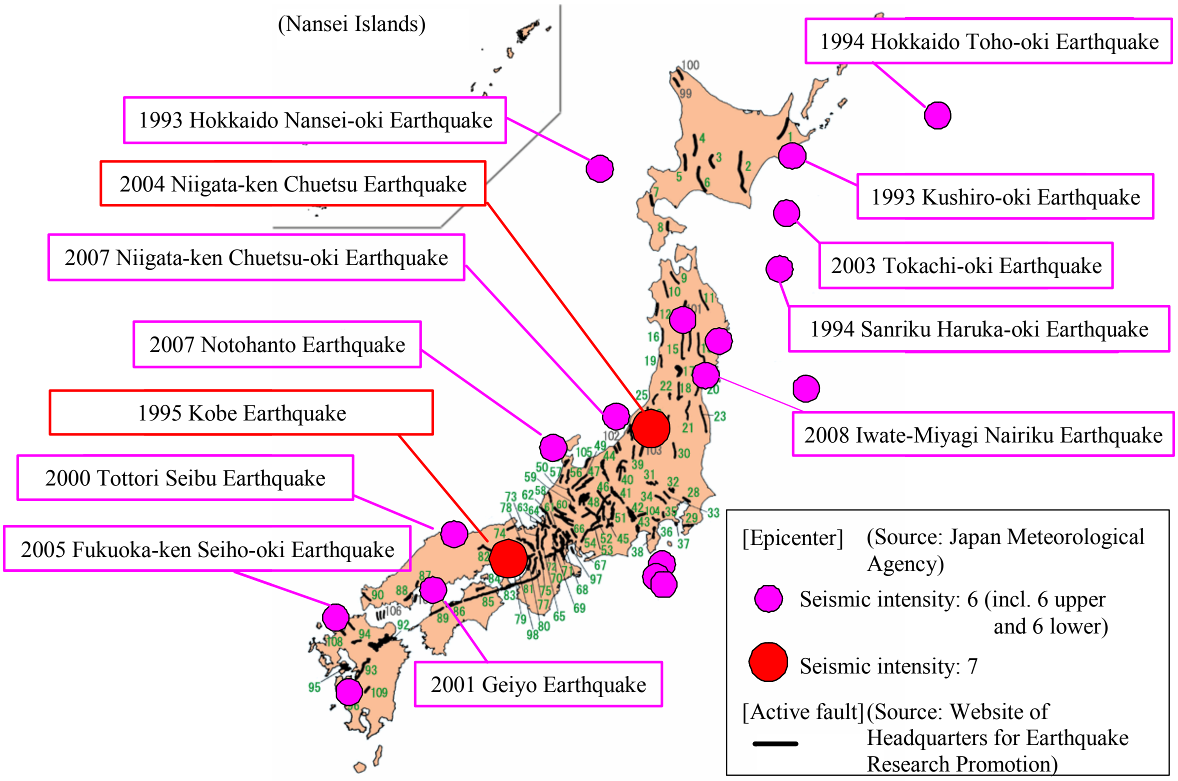
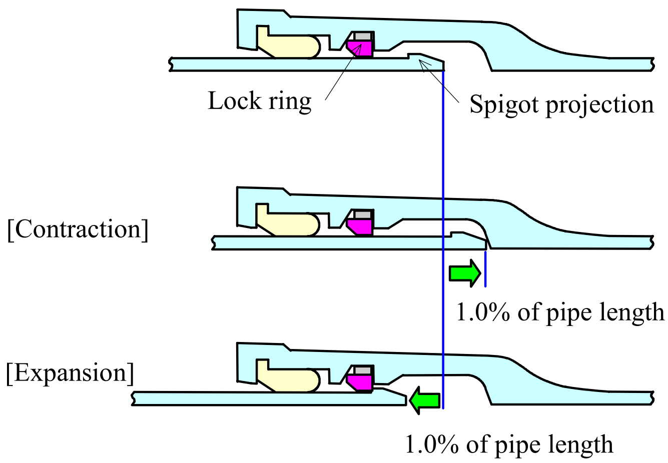
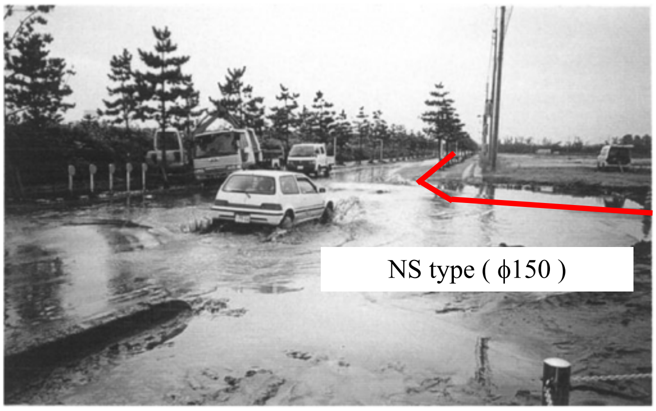


Figure 1 Areas hit by earthquakes with seismic intensity of 6 or above during and after 1993

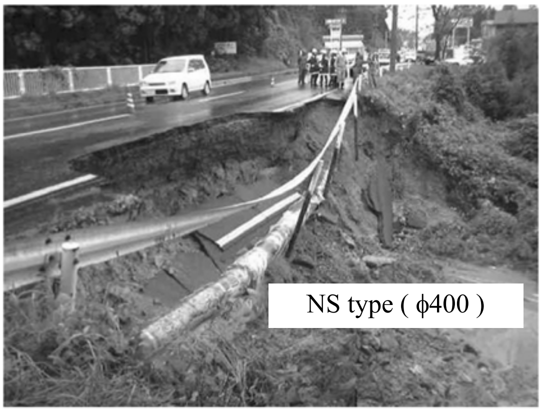


Property	Performance	ISO 16134
Amount of Expansion/contraction	±1.0% of pipe length	Equivalent to Class S-1
Slip-out resistance	3D kN (D: pipe nominal diameter)	Equivalent to Class A
Joint deflection angle	6° - 8°	—

Figure 2 Structure and performance of NS type joint



(Liquefaction)



(Ground collapse)

Figure 3 The ER-Joints withstood liquefaction and ground collapse

3. CLARIFICATION OF THE SAFETY OF THE ER-JOINTS IN LARGE SEISMIC GROUND MOTION

As described above, the Seismic Design and Construction Guidelines for Water Supply Facilities require that maximum velocity response Sv' shall be 100 cm/s for a seismic ground motion level of 2. These guidelines were established after thorough analysis of seismic waveforms recorded in the Kobe Earthquake. This velocity response corresponds to a ground strain of less than 1.0%, a level that can be completely absorbed by the ER-Joints.

In this study, we assumed extremely large seismic ground motion as high as Sv' = 300 cm/s. This is three times the level required by current guidelines.

(1) Ground Strain

For horizontally continuing stratified soil as shown in Figure 4, we determined the wavelength and ground strain on the basis of the Seismic Design and Construction Guidelines for Water Supply Facilities. The result is shown in Figure 5. For Sv' = 300 cm/s, the wavelength was determined to be in the range of 160 to 200 m, and the ground strain to be in the range of 0.9 to 1.8%. As a result, it was estimated that ground strain might exceed the expansion/contraction performance (±1.0% of pipe length) of the ER-Joints.

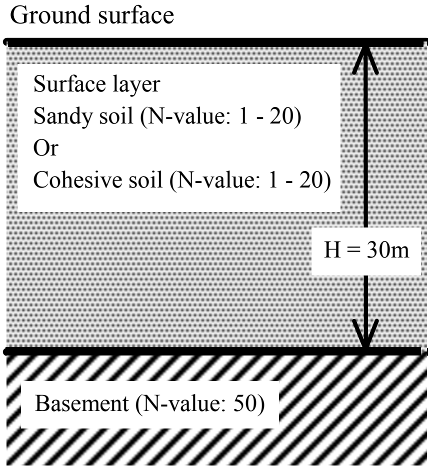


Figure 4 Ground model

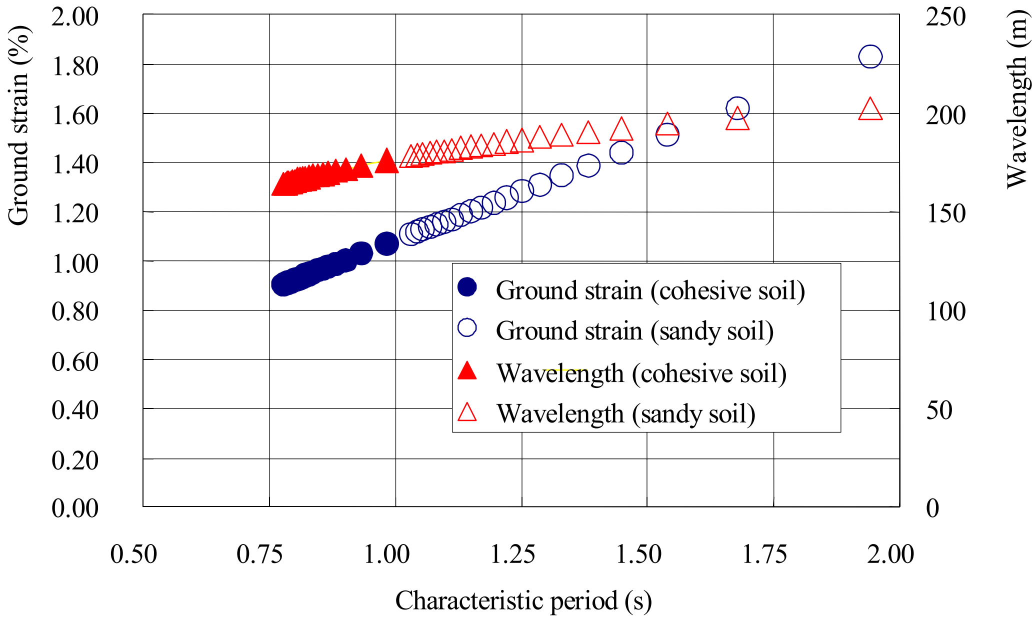


Figure 5 Wavelength and ground strain calculation results (Sv' = 300 cm/s)

(2) Mechanism of absorbing the ground strain over 1.0%

The behavior of pipeline and ground in an earthquake are schematically illustrated in Figure 6.

① When the ground strain exceeds 1.0%, the Joint A in Figure 6(a) is locked at first. Adjacent Joint B is then pulled due to the particular characteristics of the ER-Joints. The above action repeats, absorbing the ground displacement.

② If the Joint C in Figure 6(b) has not yet reached the maximum allowable expansion rate of 1.0%, it will further expand relative to the ground (in the direction of the arrow in the above figure) until the expansion rate reaches 1.0%.

③ Therefore, the ER-Joints will absorb the ground strain until area S1 becomes equal to area S2 in Figure 6(c). Here, S1 represents an area where a ground strain of 1.0% or more is generated, while S2 represents an area where the ER-Joints can expand further. Thus, the ER-Joints can absorb a maximum ground strain of approximately 3%.

④ The maximum tensile force generated by ground shaking corresponds to the frictional force between ground and a pipeline whose length is equal to a quarter of the wavelength. This tensile force is much smaller than Joint slip-out resistance (3D kN, D: pipe nominal diameter).

As discussed above, the maximum ground strain for an extremely large seismic ground motion of Sv' = 300 cm/s is approximately 1.8%. The ER-Joints can resist against this level of seismic ground motion, which is three times the level required by the current guidelines.

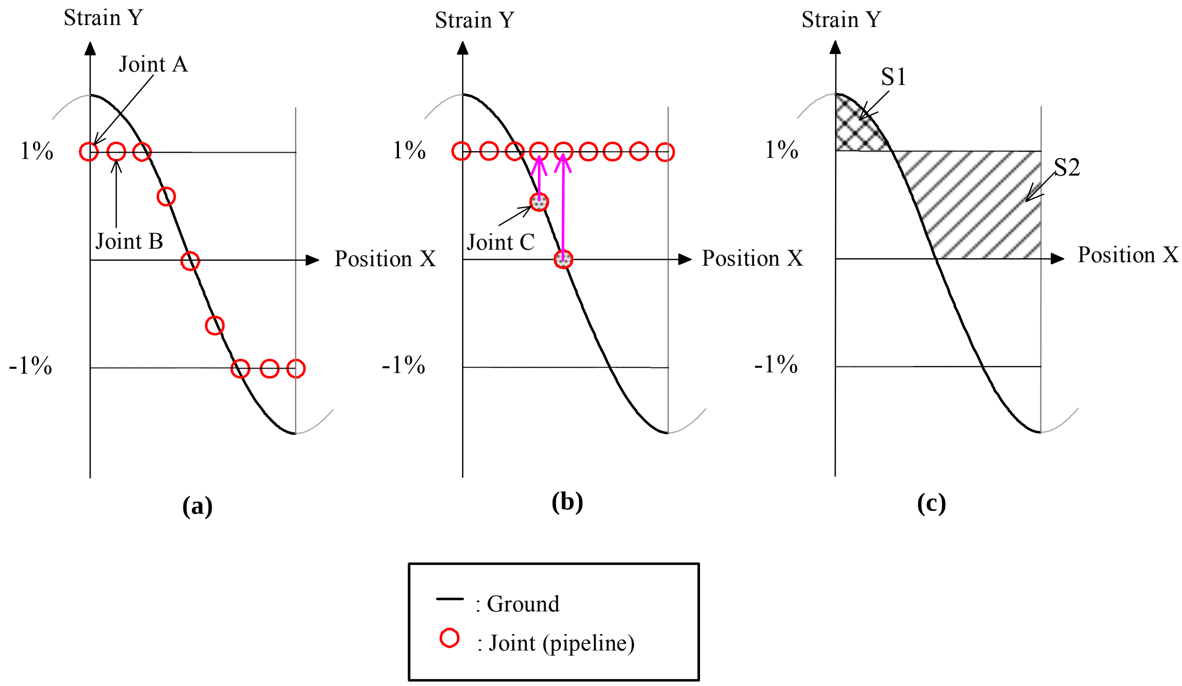


Figure 6　Behavior of pipeline and ground in earthquake

(3) Slip-Out Resistance for high speed tensile force

As the intensity of seismic ground motion increases, the speed of pipeline shaking increases, exerting high speed tensile force on the ER-Joints. If the pipeline shakes at 300 cm/s in resonance with the ground motion, the time needed to fully exhibit its slip-out resistance over a 100 m long pipeline is estimated to be 0.3 second, since slip-out resistance 3D kN is equivalent to the frictional force between the 100 m long pipe and the ground.

In this study, we applied tensile force to the test samples via impulsively for 0.1 second in order to assess their slip-out resistance.

The tensile force measurement results are shown in Figure 7. In this test, tensile force was impulsively applied to test samples having nominal diameters of 100 and 250 mm. The test sample withstood the high speed tensile force. Following the test, test samples were disassembled to check for abnormality. The results confirmed that the test samples were free from deformed lock ring, separated spigot projection or any other abnormality.

Pipe body strain measurement results are shown in Figure 8. No conspicuous difference was observed between the effects of static and dynamic tensile forces on pipe body strain. This result confirmed that the ER-Joints slip-out resistance is independent of load-exerting velocity.

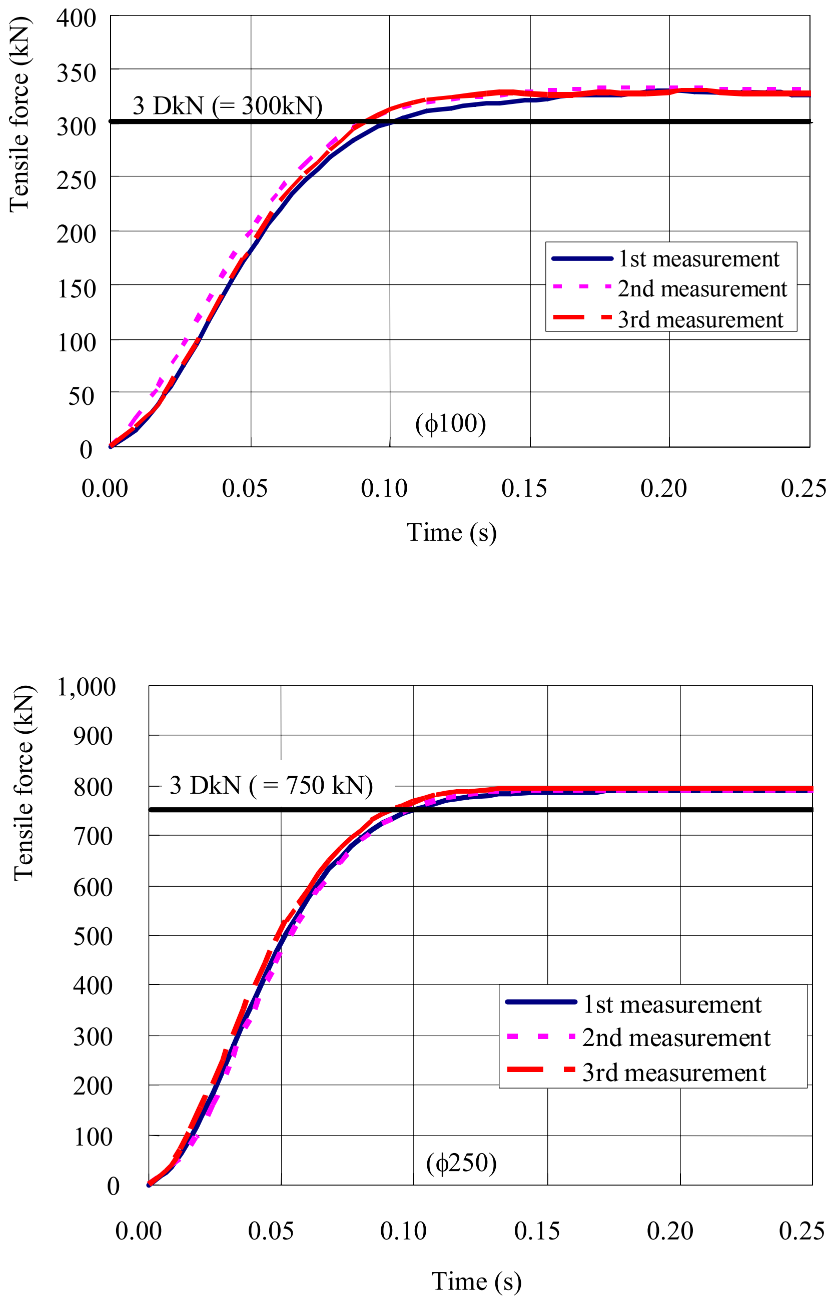


Figure 7 Tensile force measurement results (ϕ100, ϕ250)

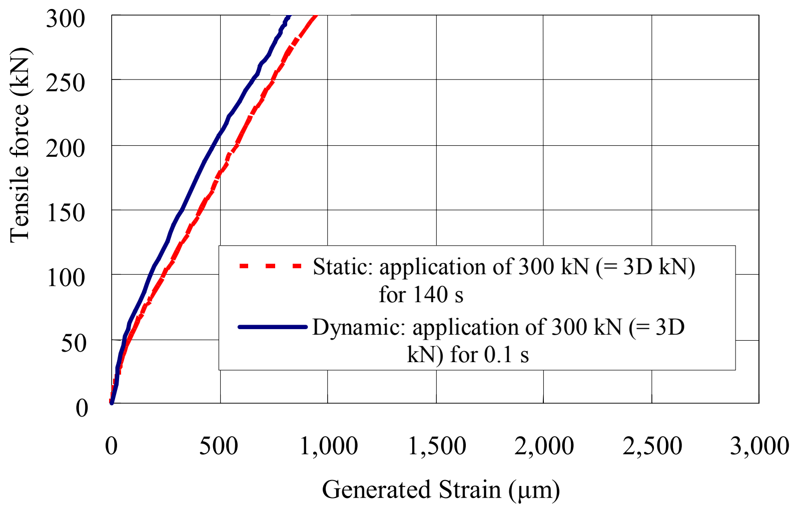


Figure 8 Joint strain measurement results (ϕ100)

4. VERIFICATION OF SAFETY AGAINST FAULT DISPLACEMENT

It is generally desirable to avoid laying pipelines crossing over a fault. But considering that there are many active faults in Japan, there are cases that we sometimes must lay pipelines crossing over a fault without any other pipeline route. We verified the safety of the pipeline against fault displacement by analyzing the buried pipeline behavior crossing over a fault in earthquake. In the conventional behavior analysis of buried pipeline, it is general practice to assume the pipeline to be a series of straight beams supported on an elastic foundation, with the pipe joints and ground connected by springs. At that time we usually assume that the direction of these springs never change even in the case of a large pipeline displacement generated by fault movement. In the present analysis, we brought in a new tool (DYNA2E), which can change the direction of these springs according to the movement of ground and pipeline.

(1) Analysis model

①Fault model

In this study, we prepared an analysis model so as to simulate the Nojima Fault that caused the Kobe Earthquake. The Nojima Fault extends approximately 10 km along the length of Awaji Island, Hyogo prefecture. It constitutes part of the Rokko-Awaji fault belt.

Kataoka et al. investigated Japanese inland earthquakes that caused faults to appear above the ground. According to their paper[3], the displacement of the Nojima Fault recorded a maximum of 2.0 m in the horizontal direction and 1.1 m in the vertical direction during the Kobe Earthquake.

On the basis of the above data, we configured a fault model that would be displaced 2.0 m in the horizontal direction and 1.1 m in the vertical direction (resultant displacement: 2.3 m), as shown in Figure 9.

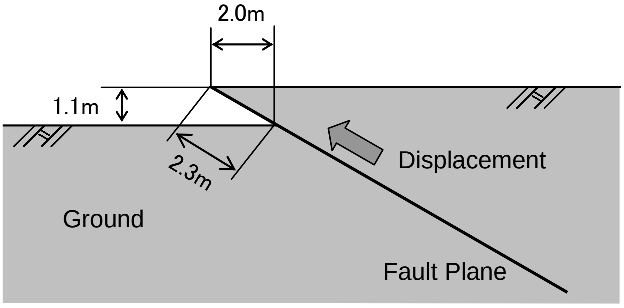
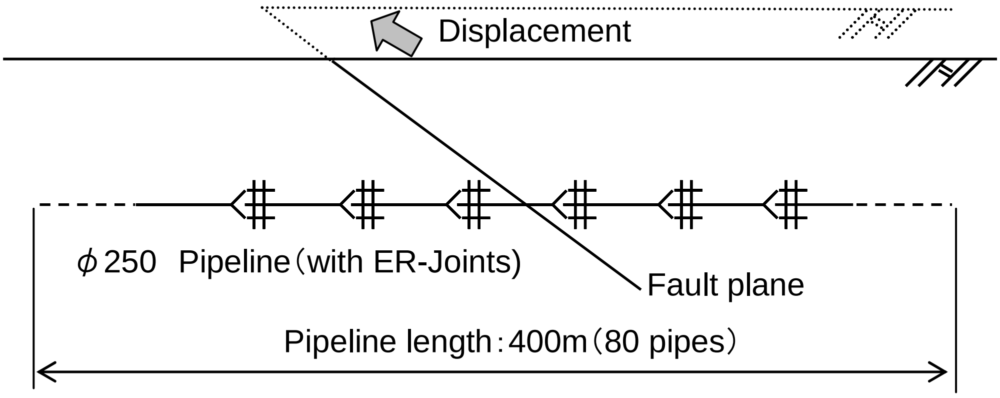


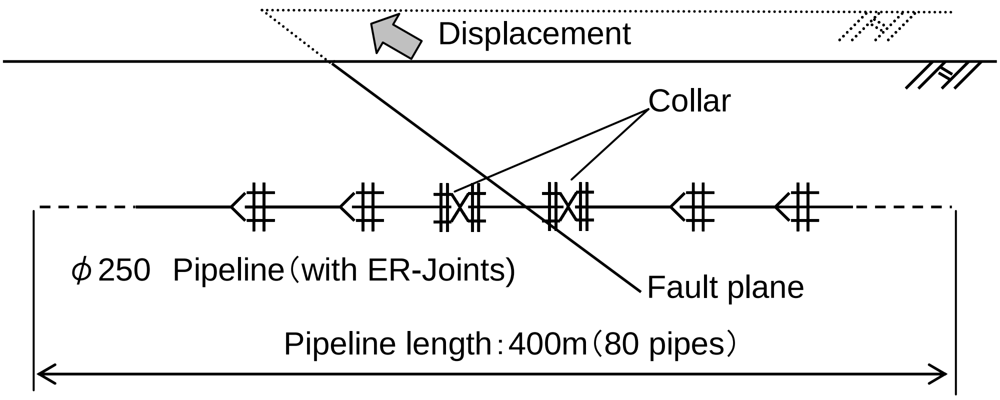
Figure 9　Fault model

② Pipeline/Ground Model

We prepared two pipeline models. One is consisted of only straight pipes (model 1). The other is consisted of straight pipes and collars which have higher deflection performance near the fault plane (model 2). Figure 10 illustrates two models in which a straight pipeline with the ER-Joints extends a distance of 400m (5 m long NS type $\phi 250 \times 80$) and crosses a fault at right angles. Each pipe and collar is assumed to be a beam supported by a ground spring; the neighboring pipe joints are assumed to be connected by a joint spring, as shown in Figure 11. Further, the joint spring is assumed to be a spring representing the expansion and contraction, slip-out resistant characteristics of the ER-Joints (Figure 12). The ground spring was assumed to be a bilinear spring representing the property of the soil material (Figure 13).



(model 1 : Only straight pipes)



(model 2 : Straight pipes and collars)

Figure 10　Pipeline model

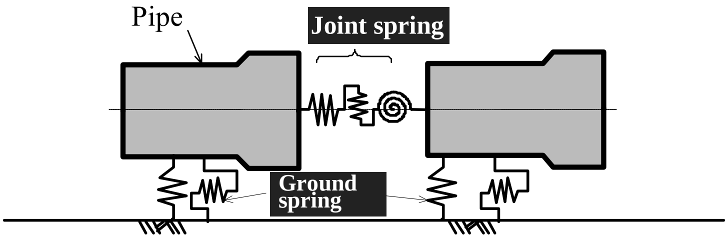


Figure 11　Joint/ground spring model

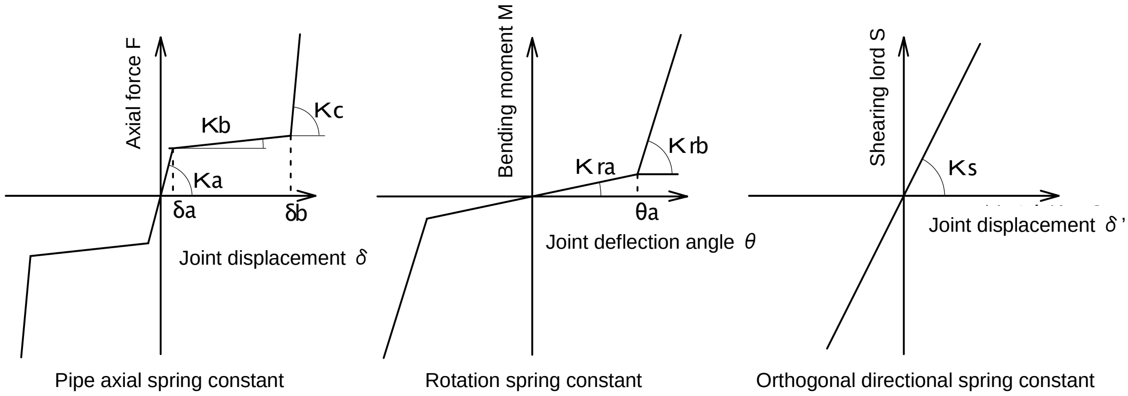


Figure 12　Joint spring constants

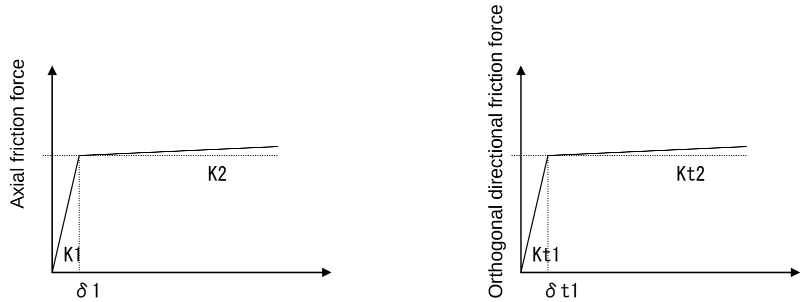


Figure 13　Ground spring constants

(2) Analysis Results

Analysis results of joint deflection angle in the model 1 are shown in Figure 14.
It is clear that deflection occurred at the nearest joints to the fault plane only and other joints showed no deflection.

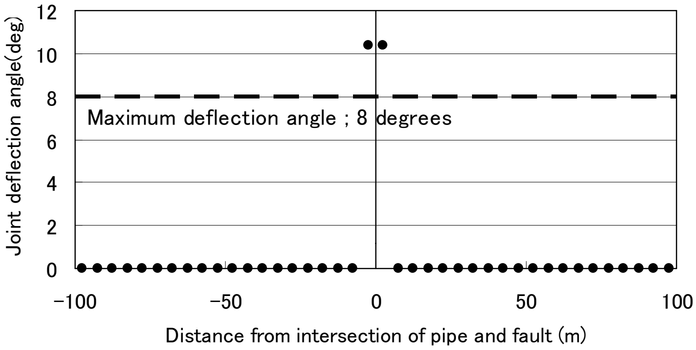


Figure 14　Analysis results of joint deflection angle (model 1)

Therefore, joint deflection angle, behavior of pipeline (vertical displacement of pipeline) and pipe body stress in the range of 20 meters on both sides of the fault plane were analyzed to observe behavior of pipeline near the fault plane in detail, and the results are show in Figure 15.

- It is clear that deflection occurred at the nearest two joints to the fault plane only and the angle was approximately 10 degrees. This deflection angle exceeds maximum deflection angle of the ER-Joint: 8 degrees.
- Pipe crossing the fault plane (Pipe B in Figure 15) was lifted diagonally according to the fault displacement, and pipelines following Pipe B, installed in the ground lifted by the fault displacement, showed parallel movement to absorb the fault displacement.
- Pipe body stress concentrated in the range of 15 meters from the fault plane, and the maximum pipe body stress was 272MPa at the intersection of pipeline and fault plane (Point A in Figure 15). This stress is equivalent to the proof stress of the ductile iron pipe: 270MPa.

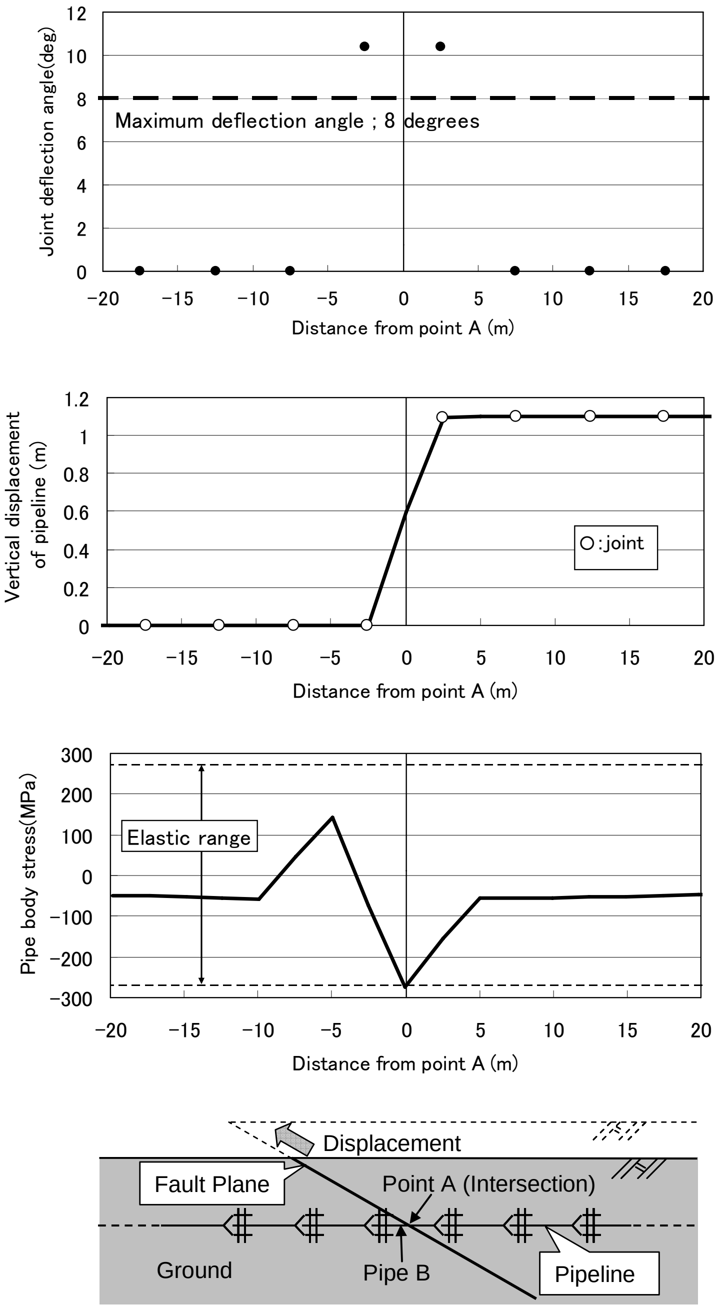


Figure 15 Analysis results (model 1 : Only straight pipes)

Analysis results of joint deflection angle, behavior of pipeline (vertical displacement of pipeline) and pipe body stress in case of model 2 were shown in Figure 16.

- It is clear that deflection occurred at the nearest two joints to the fault plane only and the angle was 6 degrees. This deflection angle is below the maximum deflection angle of the ER-joint: 8 degrees.
- Same as model 1, pipe crossing the fault plane (Pipe B in Figure 16) was lifted diagonally according to the fault displacement, and pipelines following Pipe B, installed in the ground lifted by the fault displacement, showed parallel movement to absorb the fault displacement.
- Pipe body stress concentrated in the range of 10 meters from the fault plane, and the maximum pipe body stress was approximately 160MPa at the intersection of pipeline and fault plane (Point A in Figure 16). This stress is below the proof stress of the ductile iron pipe: 270MPa.

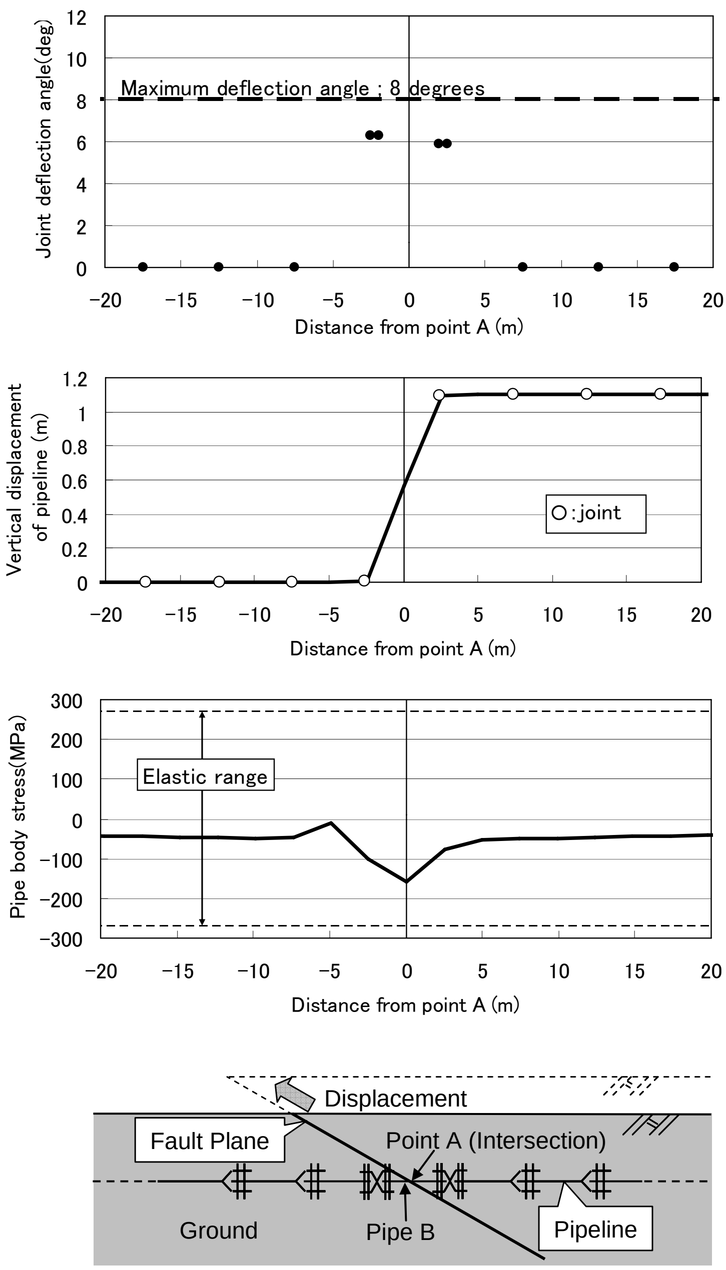


Figure 16 Analysis results (model 2 : Straight pipes and collars)

As shown above, in both cases of model 1 and 2, pipeline can absorb fault displacement of 2.3 meters by the joint deflection on both sides of fault plane. However, in case of model 1, pipe body stress is equivalent to the proof stress and joint deflection angle is larger.
Therefore, in order to improve safety of pipeline, it is desirable to make pipeline flexible by adopting fittings with higher deflection performance, such as collar.

5. CONCLUSIONS

We have studied safety of the ER-Joint resistance against a velocity response of 300 cm/s, three times the value required in current guidelines. We have also analyzed safety of the ER-Joint against the fault displacement of 2.3 m.
Results of experiment and analysis are described in the following.

1) The maximum ground strain for a velocity response of 300 cm/s is calculated approximately 1.8%. On the other hand, the ER-Joint can absorb ground strain of approximately 3% by their performance of expansion, contraction and slip-out resistance throughout the pipeline even expansion and contraction performance of a single ER-Joint is ±1% of the length of a pipe. Therefore, it is considered that the ER-Joint can absorb this level of ground strain.
2) By experiment assuming the impulsive tensile force occurred in an earthquake, safety of the ER-Joint is confirmed, as no damage occurred in the ER-Joint and pipe body stress is smaller than proof stress.
3) As a result of analysis assuming the fault displacement of 2.3m, it is confirmed that pipeline is displaced near the fault plane only and absorb fault displacement. It is also confirmed that pipeline with collar, which has higher deflection performance, is safer compared to the pipeline which consists of straight pipes only.

References

[1] Japan Water Works Association. Mar. 1997. “Seismic Design and Construction Guidelines for Water Supply Facilities.”
[2] Hisato Miura. Aug. 1996. “Investigative Study of the Behavior of Ductile Iron Pipe Earthquake-Resistant Joint in the Great Hanshin-Awaji Earthquake,” Journal of Ductile Iron Pipes (61st Edition).
[3] Shojiro Kataoka et al. Oct. 2005. “Characteristics of Surface Fault Rupture Relating to Magnitude and Slip Type of Inland Earthquakes,” Journal of Japan Society of Civil Engineers No. 801/ I-73, pp 21 - 32

The Italian Contribution to the Analysis of the Effects of Earthquakes on Water Supply Systems: Practical Experiences and Theoretical Investigations

Antonio De Risi, Maurizio Giugni, Roberto Guercio and Mario Rosario Mazzola

ABSTRACT

The recent earthquake in Abruzzi, which caused 300 victims and large-scale building destruction in the city of Aquila and the surrounding area, at a first analysis did not cause significant structural damages to the water supply and distribution systems. However an unexpected flow variation of the springs supplying the water systems of the region caused a risks of shortages in adjacent watersheds or in later periods, underlying the necessity of increasing the reliability of water systems supplied only by springs with hydrogeological watershed in earthquake areas.

The paper describes the behaviour of “Gran Sasso Mountain” springs in the last months and the possible supply alternatives for the area served by existing water systems. Furthermore the paper describes the analogous impact of the Irpinia earthquake (1980) on the springs supplying the “Sele-Calore” Aqueduct, that is the most important one serving the “Acquedotto Pugliese”, which manages water and wastewater services for more than 4 millions of inhabitants. However the major problem in that occasion has been the structural damages caused by the earthquake in the “Pavoncelli” tunnel, that is the longest one of the aqueduct. The projects carried out to overcome these problems are described in the paper.

In Italy some theoretical models of the soil-pipeline interactions have been recently developed. Such models usually assume the pipe rigidly following the soil movements caused by the seismic event or account, in more refined versions, for soil structure interaction by schematising the soil as a bed of springs according to the Winkler model which is assumed to be valid both for transverse and axial displacement. In addition, they consider infinite length pipeline and hence fail to take into account not only its effective length, but also the presence of any construction works, such as anchoring blocks, branches and so forth, which inevitably modify pipeline behaviour.

Consequently, some Italian authors have proposed a new approach, which considers a finite length pipeline with different boundary conditions at its ends. The pipeline is assumed to be continuous, i.e. any variations between the characteristics of pipe and joints are held negligible, whereas the soil is assumed linear elastic.

Many calculations were carried out, referring to end-free pipe and the end-constrained pipe. The pipe axial and transverse displacement and strain at a generic abscissa could be calculated, and therefore the stress acting on the pipe could be estimated.

Antonio De Risi, Chief Engineer Project Department , Acquedotto Pugliese, Bari, Italy
Maurizio Giugni, Full Professor, School of Engineering, Università di Napoli “Federico II”, Naples, Italy
Roberto Guercio, Full Professor, School of Engineering,, Università La Sapienza, Rome, Italy
Mario Rosario Mazzola, Full Professor, School of Engineering,, Università di Palermo, Palermo, Italy

INTRODUCTION

The Italian peninsula and the surrounding islands rank among the most seismic areas of the Mediterranean after Turkey and Greece.

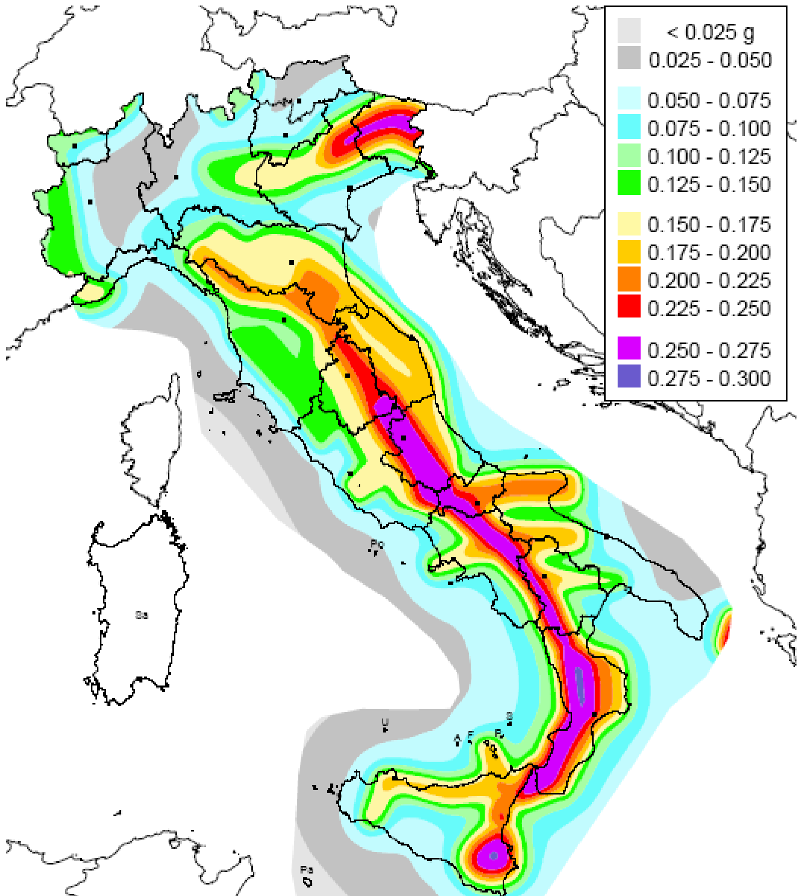


Figure 1. Peak ground acceleration (m/s^2). 10% exceedance probability in 50 years

Figure 1 shows the pattern of peak ground acceleration with a 10% exceedance probability in 50 years, a popular index of seismic hazard, for the Italian peninsula and Sicily. Values exceeding 0.25, corresponding to high hazard level, are found on the central-southern part of the Appennines and in some area of the Italian north east (Friuli).

In the last century, disastrous earthquakes include Messina in Sicily (1908) magnitude 7.2 with over 82,000 victims, Avezzano in Abruzzi (1915), magnitude 7.0 where around 32,000 were killed, Irpinia (1930) in Campania, magnitude 6.5 with around 1,400 victims, Belice (1968) in Sicily, magnitude 6.4 with 370 victims and 70,000 homeless, Friuli (1976), with 976 killed and 70,000 homeless, Irpinia again (1980) magnitude 6.9 with 2,735 killed and over 7,500 injured, and finally Abruzzo (2009) where 300 persons were killed by a 5.8 magnitude earthquake.

The paper aims at both sharing some of the practical experiences deriving from managing water resources in such a highly seismic context and informing about ongoing research in Italy on the beahaviour of buried assets under seismic conditions. The paper is hence organized as follows: the next section will describe a typical behaviour of groundwater resources after a strong eartquake in the Appenines: due to the earthquake, a displacement of aquifers and underground water bodies may take place bringing to a, sometimes drastc, change of the water supply schemes. This has occurred in the

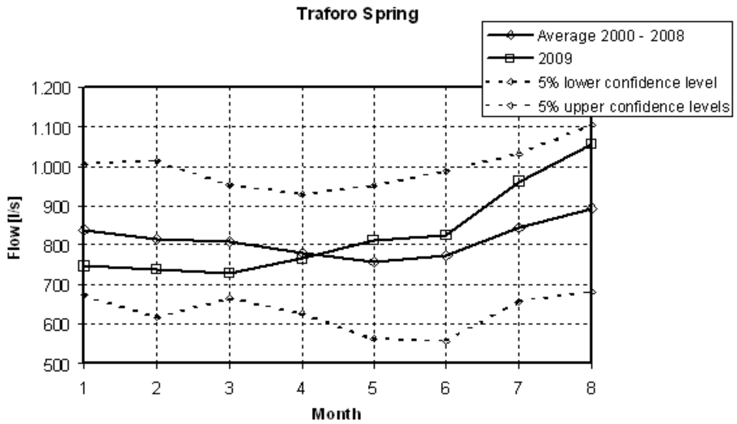
"Gran Sasso Mountain" springs in the last months after the recent Abruzzo earthquake and an analogous impact has been recorded in the case of the Irpinia earthquake (1980) on the springs supplying the "Sele-Calore" Aqueduct, the most important supply scheme for the "Acquedotto Pugliese" managing water and wastewater services for more than 4 millions of inhabitants.

The subsequent section will instead focus on some advances on the models of pipe displacement under seismic conditions.

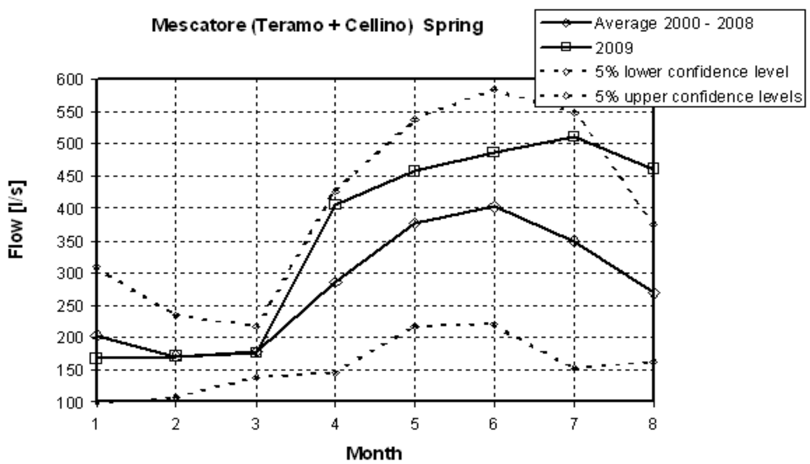
IMPACT OF EARTHQUAKES ON SUPPLY SOURCES – THE CASE OF RUZZO SPRINGS AFTER THE ABRUZZO EARTHQUAKE (2009) AND THE SELE-CALORE AQUEDUCT AFTER THE IRPINIA EARTHQUAKE (1980)

Besides their impact on infrastructures, earthquakes may also have a relevant effect on supply sources. This is frequently the case for earthquakes occurring on the Appennines. The Appennines feature large aquifers on stratified carbonatic rocks with few highly fractured areas. Overall, the hydraulic transmissivity of the aquifer is basically related to the degree of fracturation. Ruzzo springs used to constitute the main supply source for the Teramo province in Abruzzo until the construction of the Gran Sasso tunnel and the laboratories of the National Institute of Nuclear Physics (INFN) in the '80s. These large infrastructures have altered significantly the flow regime of the springs, causing an overall, consisting reduction of yield. A thorough characterization of such springs in the framework of the Gran Sasso hydrogeology is reported in [1] where geological investigation is supported by geochemical and isotopic analysis. According to the analysis, Ruzzo springs have short circuit connected to basal groundwater because, in their zone, fractured calcareous litologies outcrop alternate with either other less/not fractured litologies or with marly litologies. The connection takes places on the tectonic contact between the cretacean-eocenic carbonatic complex and the miocenic marlic bed. The activation of such tectonic contact during the recent (April 2009) earthquake in Abruzzo may be responsible for the overall increase of discharge recorded shortly after the earthquake, as documented in figures from 1 to 3. Figures 1a to 1c report the course of average monthly flows of Traforo springs (fig. 1a), Mescatore (fig. 1b) and Vecelliera Alta and Vecelliera Bassa (fig. 1c) in years 2000 – 2008 compared to the course of average flows during 2009. Mescatore and Vecelliera springs are placed at 900-1000 m above sea level and are hence the lowest of the group which also includes springs at 1200-1600 m a.s.l. However, they provide around 80% of the overall yield of the group.The figures also report 5% confidence intervals for average monthly values obtained by assuming that monthly yields are normally distributed with standard deviation as estimated by 2000-2008 data. Figures show that after April 2009 spring yield experiences an increase that keeps consistent with the average seasonal pattern of flows but flows often grows abnormally, out of the 5% confidence intervals for Mescatore and especially Ruzzo springs. Traforo spring yield grows very close to the 5% confidence upper value in august 2009, the last month on record.

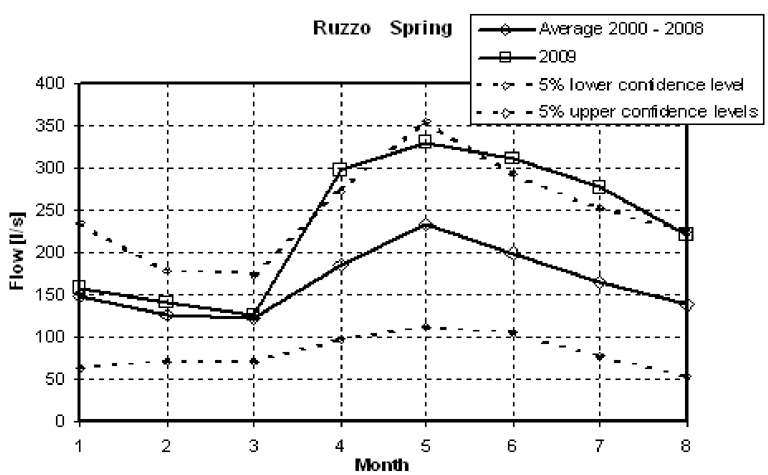
A considerable yield variation (Figure 2a and 2b) was also recorded after the Irpinia earthquake in 1980 for the Caposele and Cassano springs (around 6,0 m^3/s), both flowing into the Pavoncelli Tunnel (12 km), which constitute the main source for the Acquedotto Pugliese (Apulian Aqueduct), now supplying around 4,000,000 persons. The aqueduct., with 55 km of tunnels, was built in the first three decades of the twenieth century. Ever since its construction, the tunnel had suffered from a number of damages that had been repaired in years 1926-29 and in year 1924; in those years, repairing of some cross-section resulted in the substitution of calcareous stone with bricks both in the pillars and in the tunnel cap. The earthquake further worsened the situation: due to partial collapses of some cross-sections in the tunnel, a constant decrease in the water flow, in the order of 10 l/s per week, was observed. The risk clearly was the total interruption of water supply to Apulia, should the collapses have entirely occluded some part of the tunnel.



(a)



(b)



(c)

Figure 1. Pattern of 2000-2008 average monthly flows and 2009 average flows (squares) for Traforo (a), Mescatore (b) and Ruzzo springs (c). Dotted lines are upper and lower 5%confidence bounds.

It was then decided to build a 21 km pressurized bypass (with nominal diameter of 850 to 1050 mm) for an emergency flow of 2 m^3/s, including a pumping station for a head of 310 m. It was completed in four months and entered operations in september 1982 [2]. After securing water supply to Apulia, care was taken of the tunnel. Three different standard cross-sections were devised, depending on the level of damage: cast iron centres for different parts of the section were inserted with jets of fiber reinforced concrete.

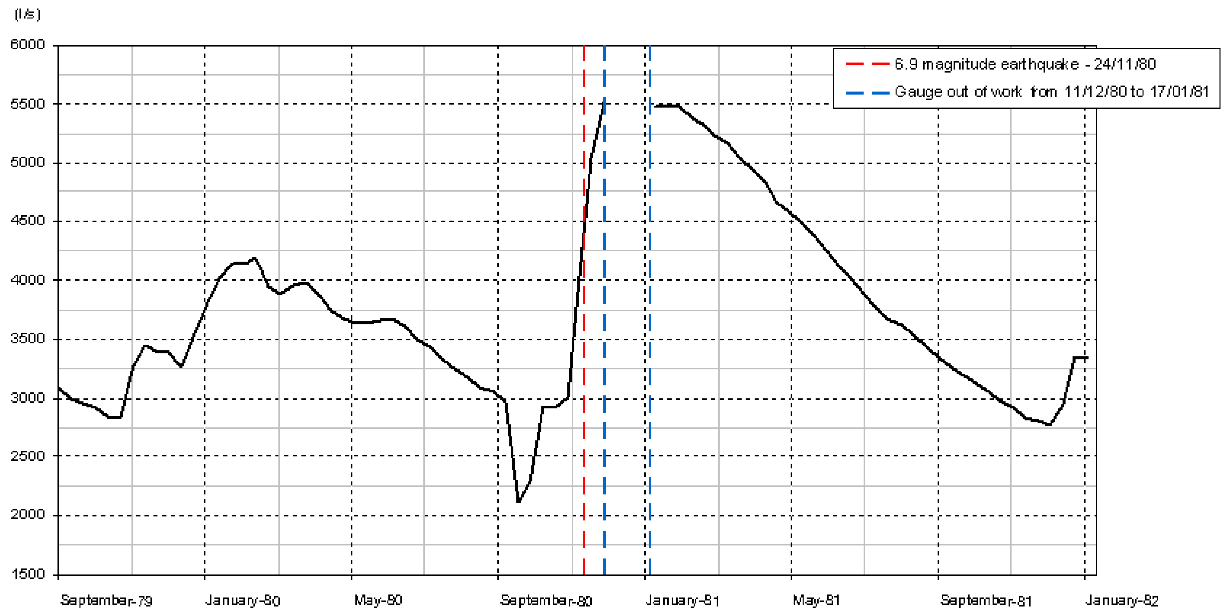


Figure 2a. Flow pattern for Cassano I spring in the time window around the 24 November 1980 earthquake in Irpinia

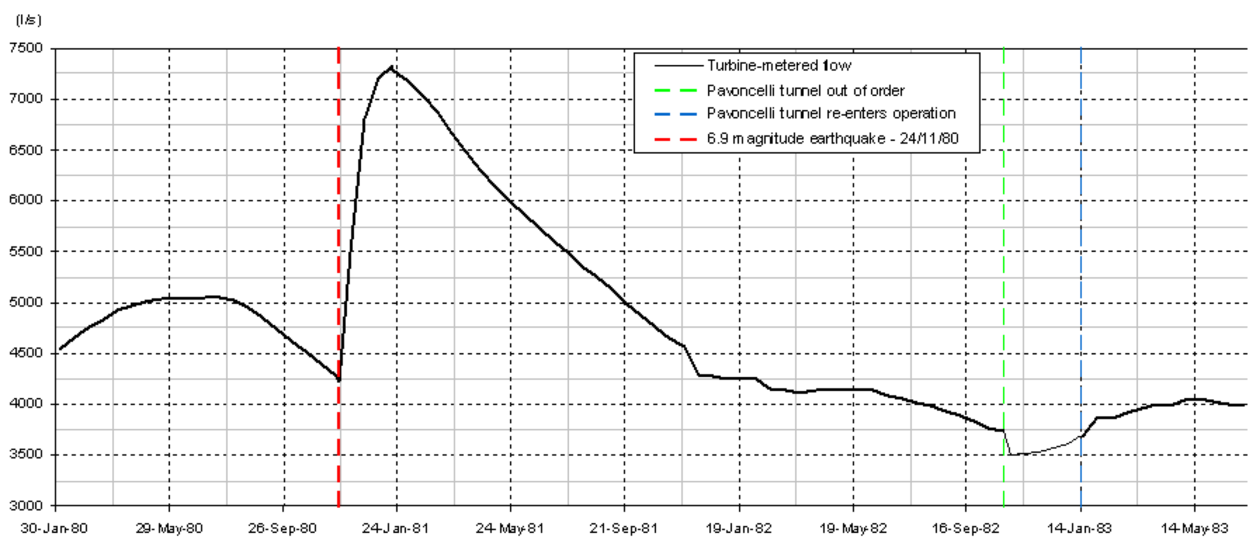


Figure 2b. Flow pattern for Caposele spring in the time window around the 24 November 1980 earthquake in Irpinia

A FINITE LENGTH BEAM MODEL OF PIPE DISPLACEMENT DUE TO EARTHQUAKES

Damage and disruption of buried pipelines caused by seismic events may have severe effects on vital lifelines, as telecommunication, water distribution and natural gas supply systems. Even when the structures are not seriously damaged, earthquakes can produce heavy loss of functionality in pipeline networks involving economic and social uneases. As a consequence, studies of seismic analysis and behavior of these structures have been presented in the literature. Investigating the dynamic behaviour of buried lifelines is a rather complex problem since it includes three dimensional dynamic analysis of the soil–structure system subject to seismic excitation. Ground – pipe interactions depend significantly on the carachteristics of both soil (depth of the layer, density,shear modulus), and pipes (elastic modulus, diameter, thickness, pipe depth) and on ground acceleration. As a result, different types of modelling of the system are developed using different degrees of simplifications with the aim of evaluating seismic stresses in the pipelines.Early works made use of a rigid model where pipe strains are the same as the ground [3] and pipe carachteristics are ignored. Such approach has also been reproposed in a (relatively) recent draft for European legislation and allows an immediate assessment of the peak axial strain and of the curvature of the pipe.

A more accurate model, incorporating the interactions between soil and structure, assumes the pipe as a beam (or a cylindric shell) immersed in a vibrating medium. In both cases, as the wave length is much bigger than the pipe diameter, the carachteristic of the seismic wave are not affected by the pipe.

If the pipe is schematized as a beam on dynamic elastic foundation and the soil as a bed of springs according to Winkler, the stress induced by the soil is directly proportional to the relative movement between the pipe and the soil (BDWF – Beam on Dynamic Winkler Foundation). This relationship is held to be valid both for transverse and axial displacement and assumes Winkler constant values according to nature and mechanical characteristics of the surrounding soil.

Under these assumptions, the equations governing axial motion of the pipe subject to the soil displacement (U_g) is:

$$-EA\frac{\partial^2 U}{\partial X^2}+m\frac{\partial^2 U}{\partial T^2}+K(U-U_g)=0$$

in which U is the pipe displacement in X direction (longitudinal direction), assumed the same as wave propagation, m the mass per unit length, E the Young’s modulus of the pipe material, A=πs(DE-s) the area of the cross section, where s and DE are the thickness and the external diameter of the pipe respectively, and K=kπDE, in which k is soil’s Winkler constant.

Assuming an elastic soil–pipe interaction, the BDWF model allows to evaluate pipe strain and hence pipe stress produced by soil movements during seismic events.

Following this hypothesis, various authors consider infinite length pipeline and hence fail to account for its effective length and any construction works, such as anchoring blocks, branches, manholes and so forth, which inevitably modify the pipe behavior. These differences can be immediately inferred if we consider a water distribution or drainage network where branches, shafts or chambers can be expected at much smaller intervals; but they also apply to external water supply pipelines where construction works can be envisaged, at most, every few hundred metres. Consequently, a new approach was suggsted [4], [5], which schematizes the dynamic behavior of a finite length pipeline with different boundary conditions at its ends (FLBDWF – Finite Length Beam on Dynamic Winkler Foundation).

The pipeline is assumed to be continuous, i.e. any variations between the characteristics of the pipe and those of the joints are held to be negligible, whereas a linear elastic soil model was assumed and no slippage at the pipe-soil interface was considered. Since the end constrains of the pipe influence the seismic response, two different boundary conditions were considered: free ends and soil-constrained ends. The first assumption is sufficiently close to the pipe behavior when the constraints are such as to allow unrestrained deformation. The second boundary condition consists in assuming that the constraints at either end of the pipe are such as to prevent all relative movement between the construction works and the pipe. Obviously the proposed boundary conditions are to be considered like borderline cases, only partially corresponding to the effective physical behavior of the oil-pipe system.

The authors [4] [5] carried out numerical simulations referring to steel and HDPE pipes with free and constrained ends in order to identify the parameters that influence pipe's seismic response. The effects of soil, pipe length and stiffness pipe on the seismic behavior were analysed emphasizing that the constraining condition at the pipe ends strongly affects its dynamic response.

Numerical results, now limited to axial strain, are shown in figure 2. In the graphs, the ratio R Umax/Ugmax between maximum (axial) pipe displacement and maximum ground displacement is plotted as a function of the pipe length in the case of steel pipe (figure 2a, on top) with Dn = 500 mm and in the case of high density PE with Dn = 500 mm (figure 2b, bottom). In both cases, V_s, the propagation velocity through the soil, was set to 200 m/s.

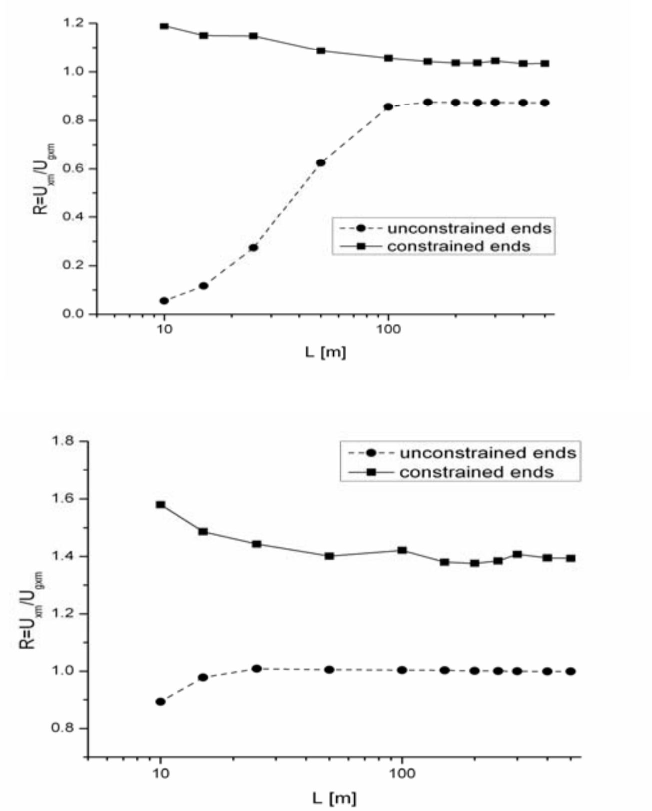


Figure 2. Maximum axial pipe displacement / maximum ground displacement ratio versus pipe length for steel (upper panel) and HDPE (lower panel) pipes

Results show that for free ends pipe a BDWF (or rigid) model can be applied: both the maximum axial displacement and the maximum axial strain of the pipe are similar to the soil one. For constrained ends pipes the rigid model under-estimates the axial strain. According to the proposed approach, the maximum pipe strain strongly exceeds the soil one, particularly for HDPE pipes, for which short lengths attain greatest strains and hence the FLBDWF model could better estimate the pipe seismic stress.

Further results, obtained considering real ground motion excitations, follow the pattern calculated according to a sinusoidal soil motion, although the latter produces pipe strains always greater than actual earthquakes. Results of such simulations are reported in figure 3 (steel and HDPE with unconstrained ends) and in figure 4 (steel and HDPE with constrained ends).

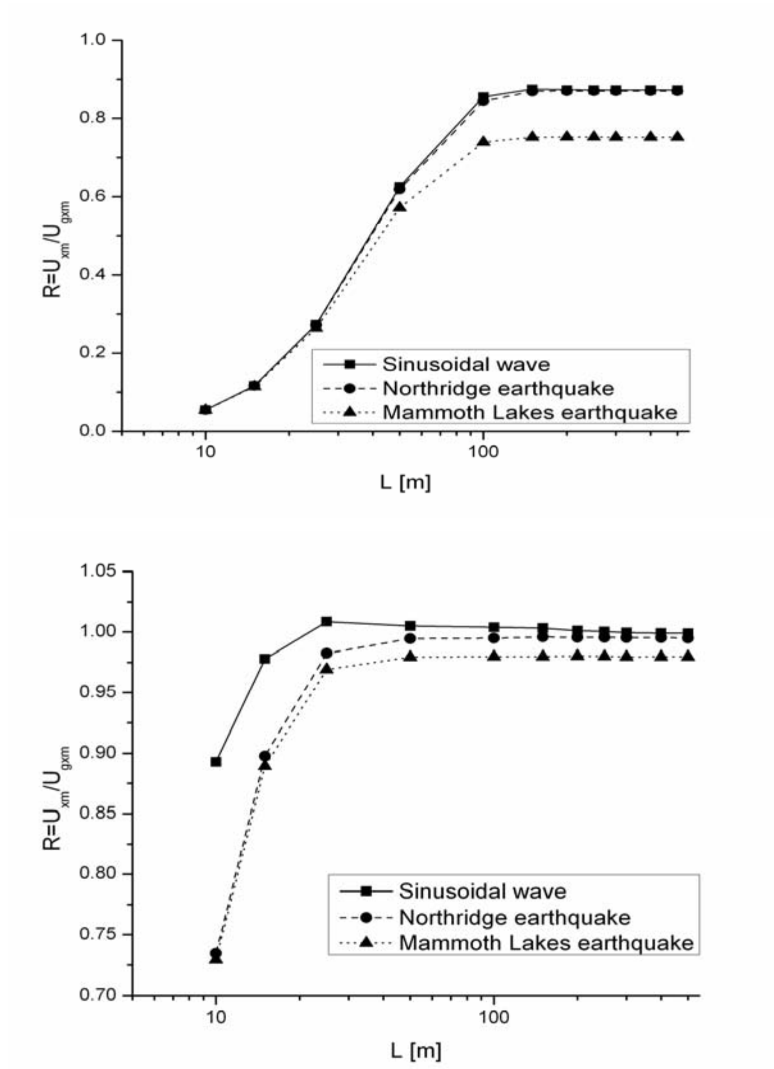


Figure 3. Maximum axial pipe displacement / maximum ground displacement ratio versus pipe length for steel (upper panel) and HDPE (lower panel) pipes under different seismic loads with unconstrained ends

Figures 3 and 4 show that the R ratio behaves differently for short pipe length (< 100 m) according to whether pipe ends are constrained or unconstrained. In pipes with unconstrained ends, R increases with pipe length, whereas in pipes with constrained ends, R decreases with pipe length.

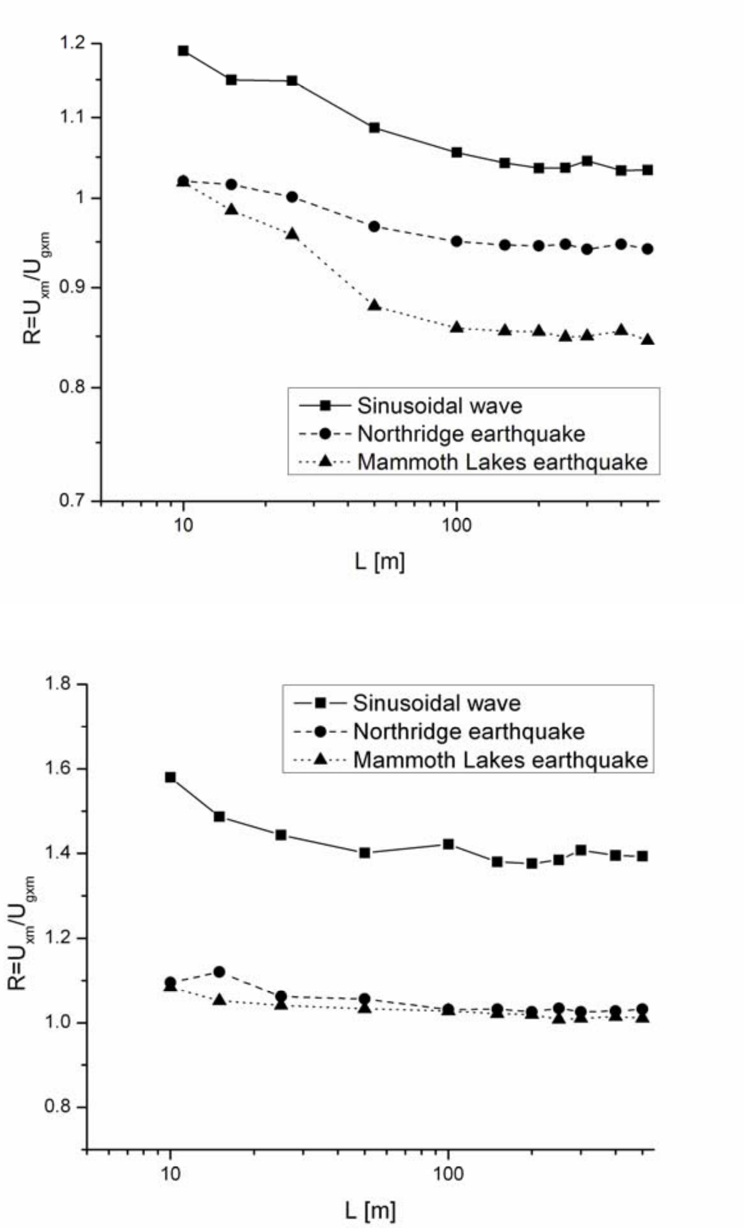


Figure 4. Maximum axial pipe displacement / maximum ground displacement ratio versus pipe length for steel (upper panel) and HDPE (lower panel) pipes under different seismic loads with constrained ends

CONCLUSION

The paper has illustrated some of the pratical issues in water supply that have arosen in southern-central Italy after two very relevant earthquakes and the result of a theoretical model of pipe displacement under seismic conditions proposed by Italian authors. As trivial as it may sound, the main lesson learnt from the two case studies is that a reactive appoach in water supply issues after an earthquake may probably not be the best one: although earthquakes are the typical class of natural phenomena that are considered so catastrophic that water supply issues unavoidably shift in the background, advances in seismical design of buildings and infrastructures may make unadequately designed water supply systems one of the bottlenecks of the recovery from serious earthquakes. In other words, earthquakes should be aknowledged as one of the sources of risk in water supply and an appropriated risk-based approach design of supply systems shoud be introduced. Risk here is related not only to the structural failure of tunnels or large buried assets, but also to significant variations of the supply schemes due to changes in aquifer configuration.

REFERENCES

[1] Petaccia, R., and S. Rusi. 2008. "Idrogeologia delle sorgenti del Ruzzo (Gran Sasso d'Italia)", Giornale di Geologia Applicata 8(1):17 – 28 doi: 10-1474/GGA.2008-08.1-02.0212.

[2] Beomonte, M., Rotondi, L., Consiglio, G., and D. Trulli. 1986. "Gli interventi urgenti nella galleria di valico di Caposele dopo il sisma del novembre 1980", *International Congress on large underground openings,* Florence, Italy, 46 – 50.

[3] Newmark, N.M., 1967. "Problems in Wave Propagation in Soil and Rocks," *Proceedings of International Symposium on Wave Propagation and Dynamic Properties of Earth Materials*, Albuquerque, NM, pp. 7-26.

[4] Corrado, V., B. D'Acunto, N. Fontana, and M. Giugni. 2008. "A new approach for the analysis of dynamic response of continuous buried pipes in seismic areas", *Journal of Transportation Engineering*, (in revision).

[5] Corrado, V., B. D'Acunto, N. Fontana, and M. Giugni. 2009. "Estimation of dynamic strains in finite end-constrained pipes in seismic areas", *Mathematical and Computer Modelling*, (49): 789-797, 2009.

Estimation of Seismic Resistivity and Earthquake-resistant Measures of Water Distribution Facilities in the Hachinohe Regional Water Supply Authority

Noboru Murakami

ABSTRACT

The Hachinohe Regional Water Supply Authority has been promoting preparation of seismic resistivity of pipes and facilities and emergency work system after earthquakes（the Tokachioki Earthquake in 1968 and Sanrikuharukaoki Earthquake in 1994）.

The present main water pipes and facilities are seismic resistant and are centralized such that water is transmitted from Hakusan-Water-Purification-Plant to about 30 reservoirs in wide service area. However, many old distribution pipes and small facilities constructed before 1970 are still not seismic resistant.

For estimation of seismic resistivity of these pipes, quantitative analysis such as seismic damage forecast simulation for earthquake-supposed; estimation of number of days for service-start after earthquake; estimation of aged pipes; check of distribution-pipe route to important institutions were carried out.

As a result of these analysis, different improvement procedures were recognized. For example, the extent of damage of ＰＶＣ is larger compared with other kinds of pipes.

Therefore, it is necessary that efficient and effective earthquake-resistant measures should be promoted according to the importance of pipes for function improvement.

Noboru Murakami, Assistant Manager of Management Planning Division, Hachinohe Regional Water Supply Authority, 1-11-1 Minami- Hakusandai, Hachinohe, Aomori Japan 039-1112

1. INTRODUCTION

Hachinohe Regional Water Supply Authority with a core of Hachinohe city is a water utility with wide area established by integrating ten (10) water suppliers in 1986. The scale of operation is as summarized consisted of: i) water supply area with 800km^2; ii) population served with some 33,300 heads; iii) numbers of customers with approximately 134,000 households; and iv) maximum daily water supply with 106,770 m^3.

This area was affected by plate-trench type earthquake because she is located in the south-eastern part of Aomori prefecture opens onto the Pacific Ocean. In 1968, water supply pipelines were heavily damaged by Tokachi-oki Earthquake (M_J 7.8). Promotion on seismic resistivity of water supply facilities and improvements on emergency work system after earthquake disaster had been pursued so far by learning from such experience.

On the other hand, that the concentrated pattern centered on Hakusan Water Treatment Plant (hereinafter referred to as “WTP”) was applied to be changed from the distributed pattern with twenty-one (21) WTPs resulted from discontinuation and/or expansion program on small-scale intake and purification facilities after the authority’s establishment.

However, many distribution pipes installed before 1970 were still without seismic resistivity. The rehabilitation for the distribution pipes those may cause incident leakage accident even under normal conditions can le to the strengthening on earthquake disaster preventative measures.

Accordingly, seismic resistivity on water supply pipelines was rated by conducting the damage forecast on earthquake-supposed. In addition, field study on corrosion/deterioration of water supply pipelines was carried out and rated as the deteriorated degree. Figures by rating on each block distribution system were applied to such evaluation. Moreover, present conditions on seismic resistivity were understood by searching the distribution routes to the schools and hospitals where become critical site facilities in the case of earthquake disaster and are summarized is as below.

2. WATER SUPPLY FACILITIES AND MEASURES AGAINST EARTHQUAKE

1) Outline of Water Supply Facilities

There exist three (3) water treatment plants consisted of: i) Kanisawa WTP with water source originated from springs and groundwater; ii) Mishima WTP with water source originated from springs and groundwater; and iii) Hakusan WTP with water source directly from surface water of the Mabechi River. There are forty-one (41) distribution reservoirs in total, and the treated water was conveyed by Hakusan WTP up to the thirty-seven (37) distribution reservoirs forming as an arborscent pipeline system. Transmission pipes were already improved by quake-resistant pipes (S-type & II-type) since 1989.

Final effluent by Yomasari Dam of the Niida River system as a new water source will be purified by Hakusan WTP from April 2010 scheduled. This may ensure that mutual accommodation on raw water drawn from the Mabechi River as the existing water source becomes possible and thus backup system can be formulated in case of emergency. In addition, the water conveyance pipes from both rivers were constructed by the welded steel pipes with diameter of 1,200mm (thickness of 16mm) for the Mabechi River system and by the ductile iron pipes (diameter: 1,000mm) with seismic flexible joint (Type S) for the Niida River system.

With regard to the water-distribution management, distributing area for the said forty-one (41) distribution reservoir systems was subdivided into 143 block distribution systems for operation.

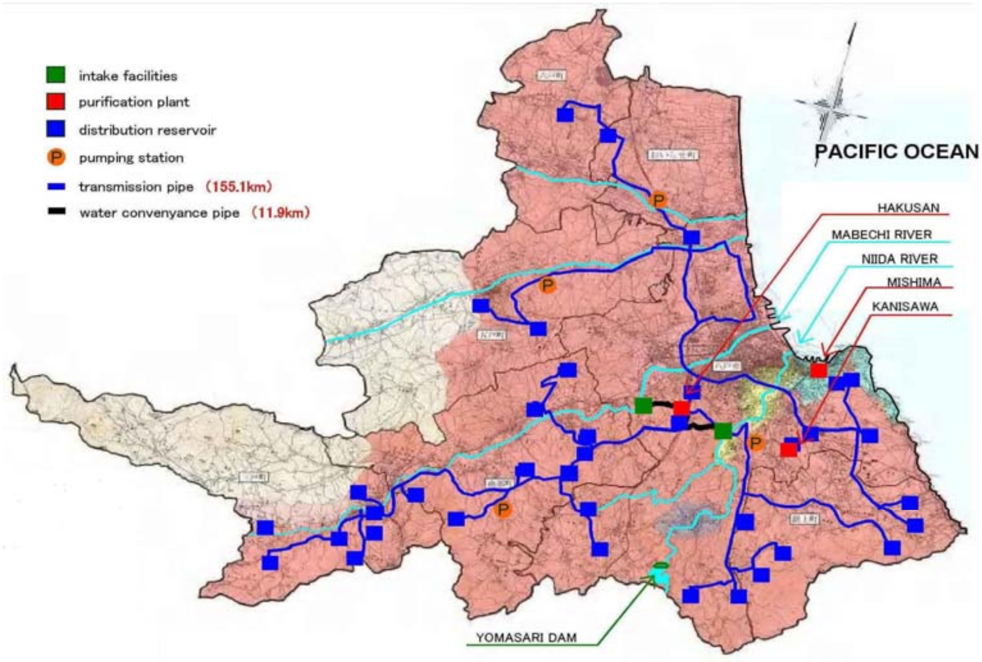


Figure-1 Outline of Water Supply Facilities

2) Seismic Resistivity on Structures

Structures shall be constructed with seismic resistivity after the revision on seismic resistivity standards by "Seismic Resistivity Construction Methods/Guidelines on Water Supply Facilities (1997 version)". In addition, seismic diagnosis was conducted for the existing major structures to perform the reinforcing and/or rehabilitation works. Table-1 shows the quake-resistant facilities for the Hakusan WTP system as the major facilities.

Table-1 Quake-resistant Facilities for Hakusan WTP System

Name of Facilities	Construction Year
WTP Administrative Building	2005 (Reinforcement)
WTP (Capacity by the Niida River System: 50,000m^3/day)	2006 (Reinforcement)
Intake Pump Station	2006 (Reinforcement)
Distribution Reservoir (Volume: 10,000m^3)	2007 (Reinforcement)
WTP Rapid Sand Filter	2008 (Reinforcement)

3) Seismic Resistivity on Pipelines

Water conveyance and transmission pipes improved by expansion program since 1989 were constructed by quake-resistant pipes (S-type & SII-type). All distribution pipes were constructed by quake-resistant pipes because those pipelines were damaged by Sanriku Haurkaoki Earthquake in 1994. In addition, construction by polyethylene sleeve cover with an effect on pipe corrosion prevention to increase in longevity of pipes was thoroughly carried out since 1991.

4) Emergency Work System

Emergency water supply bases were planned to be improved by installing five (5) emergency water tanks (total capacity of $240m^3$) and forty-eight (48) emergency feed valves. As a future policy, emergency water tanks shall be improved by quake-resistant pipelines and more emergency feed valves were slated to be installed to this pipeline. Equipment and materials necessary for emergency water supply and emergency restoration work were distributed and equipped to the respective basic water supply facilities of three (3) areal blocks (Hachinohe, Okuirise and Mabechi) to promptly respond the earthquake disaster. With regard to the backup and cooperation system, agreements were ratified with water suppliers or local industry segments (water supply business, construction business, electrical/mechanical works business) etc. against the seismic hazard. In addition, supply system in case of emergency was improved by connecting the distribution pipes with the adjacent water suppliers even a few water volumes. Furthermore, standby generators were prepared within WTPs and pump stations in case of power failure.

3. EARTHQUAKE-SUPPOSED DAMAGE FORECAST AND ESTIMATION OF SEISMIC RESISTIVITY OF DISTRIBUTION PIPELINES

3-1 Earthquake Scale and Forecast Approach

Earthquake-supposed was targeted to the assumed Pacific Ocean trench obtained from "The Study Report on Damage Forecast by Earthquake and Tsunami in Aomori Prefecture (March 1997)". Pipeline damage forecast was simulated by inputting such relevant data on earthquake ground motion into the respective pipeline attribute data of mapping system. In addition, damage forecast aimed at the distribution pipes with about 2,200kms in length and diameter over 50mm.

Damage forecast equation proposed by "Earthquake Damage Forecast on Water Supply Pipelines (November 1998), Japan Water Works Association" was adopted for damage forecast. Four (4) modification factors on pipe type, diameter, morphogenetic ground, and liquefaction risk are set up for the said equation. First, damage rate (case/km) was estimated by the said equation applying these factors mentioned above. Next, several damage models were created from the estimated damage rate and water suspension/water-reducing ratio (%) were thus estimated by Monte Carlo simulation method. Finally, recovery days were estimated from damage counts and water suspension/water-reducing ratio calculated.

3-2 Relevant Data on Earthquake Ground Motion

Earthquake ground motion with 240-gal, 380-gal, and 675-gal respectively was set up by modifying the seismic intensity scale with 5 lower, 5 upper, and 6 lower respectively estimated into the ground acceleration. Figure-2 illustrates the seismic distribution map.

Risk was classified into four (4) ranks by using the PL value of liquefaction risk coefficient. In addition, morphogenetic ground was partitioned by 250-mesh unit on the map and finally that with higher area ratio was thus applied among of these meshes.

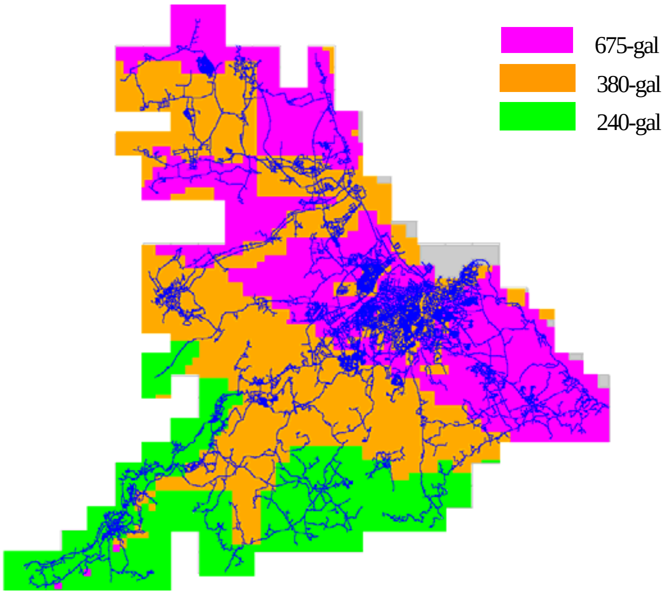


Figure-2 Seismic Distribution Map

3-3 Predicted Outcome

1) Damage Level and Damage Count

Average damage rate (case/km) was estimated as 0.31 within the whole water supply area. Damaged counts were estimated as 708 cases in total consisted of: i) 22 cases on trunk pipeline with diameter over 300mm; and ii) 686 cases on distribution pipeline with diameter blow 250mm. With regard to the individual block distribution system, average damage rate was 1.13% maximum and damage counts were 87 cases.

Comparing with Sanriku Harukaoki Earthquake on December 1996, damage with approximately five times of the same was forecast as summarized on Table-2.

Table-2 Comparison between Earthquake-supposed Caused by Pacific Ocean Trench and Sanriku Harukaoki Earthquake in 1996

Name	Max. Acceleration (gal)	Average Damage Rate (case/km)	Damage Count (case)
Earthquake-supposed Caused by Pacific Ocean Trench	675	0.31	708
Sanriku Harukaoki Earthquake	602	0.06	113

Damage rate on water supply pipeline shows high along the costal area of Pacific Ocean with high ground surface acceleration and liquefaction risk as illustrated in Figure-3. In addition, hard-hit damages were forecast within Hachinohe city occupying 70% of whole population served.

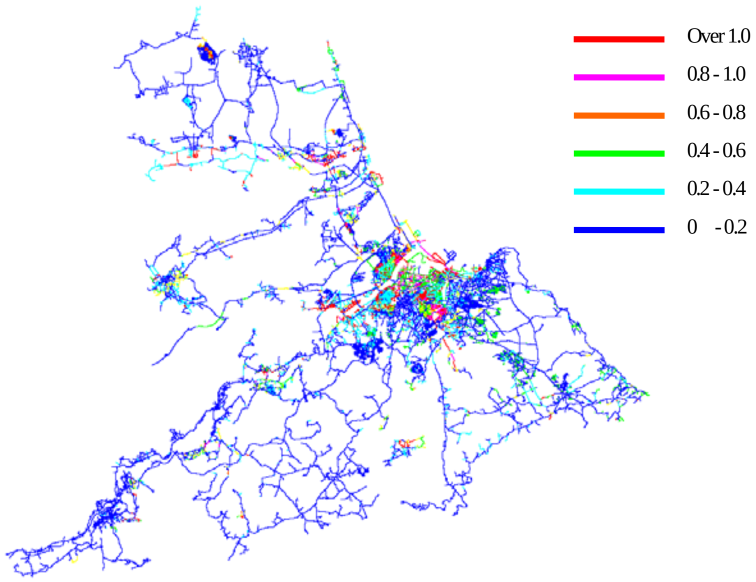


Figure-3 Distribution Map on Pipeline Damage Rate

Average damage rate on DIP except quake-resistant joint pipes shows the minimum with 0.17 in term of pipe type; however, damage counts reached 277 cases since it almost occupied some 50% of total pipeline length. On the other hand, average damage rate on VP shows high as 0.76 and damage counts reach the maximum with 315 cases. (See Table-3)

Table-3 Damage Rate and Damage Count by Pipe Type

Pipe Type	Average Damage Rate (case/km)	Damage Count (case)
ACP	0.49	30
CIP	0.51	9
DIP*	0.17	277
PP	0.33	58
SGP	1.08	14
SSP	0	0
VP	0.76	315
Others	0.63	5

Note: DIP excludes the quake-resistant joint pipes

2) Ratio and Population Served by Water Suspension/Water Reducing

Hydraulic analysis was carried out by defining the pipeline that effective he is below 10m under poor running water condition (water pressure at node of pipeline network below 0.098Mpa) as "water suspension/water reducing". Consequently, ratio by water suspension/water reducing was 22.3% and populations served by water suspension/water reducing were as estimated approximately 74,200 heads within the whole water supply area.

3) Recovery Days

Twenty-five (25) restoration work teams were estimated to resume the damage counts with 708 cases. Consequently, recovery period was required as thirty-seven (37) days in total consisted of: i) two

(2) days for trunk line with diameter over 300mm; and iii) thirty-five (35) days for distribution pipes with diameter below 250mm.

3-4 Seismic Resistivity by Figures

Ratio on water suspension/water reducing for each block distribution system estimated by earthquake-supposed damage forecast was evaluated as the rating item. Rating grades was estimated between the highest grade and the lowest grade by linear equation applying 100% of water suspension/water reducing ratio as the lowest grade (one point) and 0% of water suspension/water reducing ratio as the highest grade (100 points) respectively. Figure-4 illustrates the rating results on block distribution systems within Hachinohe area to be forecast with hard-hit damages.

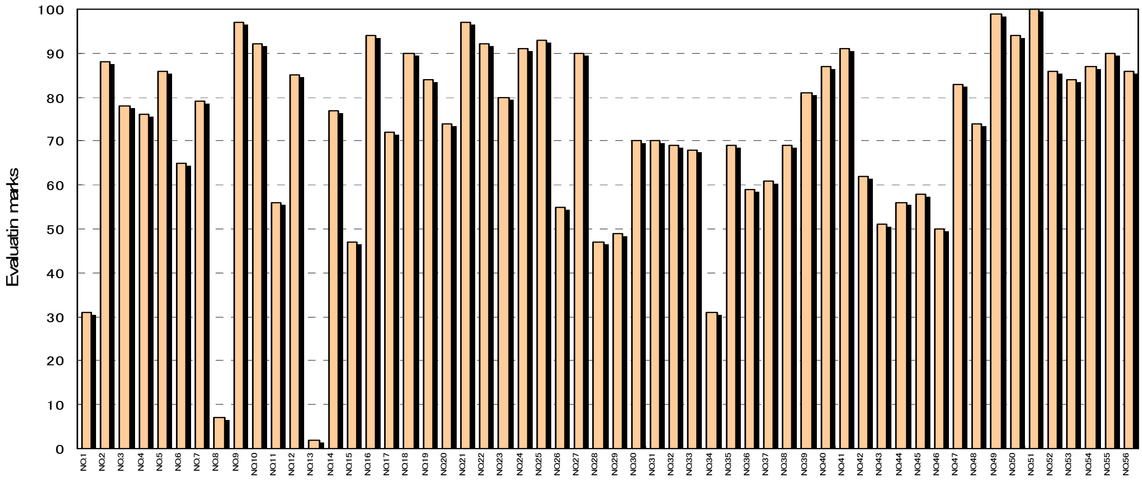


Figure-4 Seismic Resistivity Rating (Block Distribution System within Hachinohe Area)

4. DETERIORATED DEGREE RATING

4-1 Present Condition on Water Supply Pipeline

Total length on water supply pipelines are approximately 2,200kms (diameter with over 50mm), and consists of about 110kms trunk pipeline with diameter over 300mm and about 1,900kms distribution pipes with diameter below 250mm. With regard to the major pipe type, DIP equipped with both quake-resistant joint and ordinary joint occupied about 75% (about 1,650kms), VP occupied about 16% (about 350kms) and PP occupied about 7% (about 150kms) respectively.

With regard to the yearly installation length on water supply pipelines, installation length tends to rapidly increase since 1964. This is implying the conditions that pipelines were being renewed before 1963. However, pipeline installed before 1969 passing over forty (40) years of legal useful life with approximately 180kms in total length are still remaining. (See Figure-5)

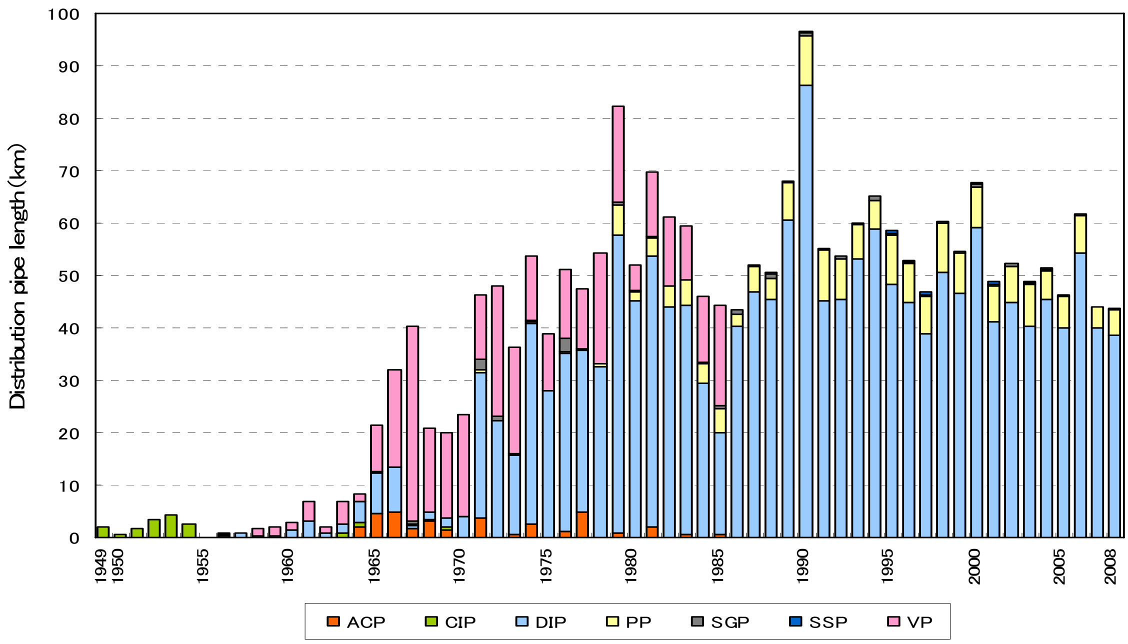


Figure-5 Year-on-year Pipeline Length (before 2006)

4-2 Study on Corroded/Deteriorated Degree

Deteriorated degree on the pipeline is influenced a great deal by the soil surroundings underground. Judgment on deteriorated degree only by the years buried cannot tie in with accurate evaluation taking this fact into account. Therefore, prospecting survey was conducted for those 214 sites where both DIP and CIP without polyethylene sleeve cover were already installed. Rating on deteriorated degree is judged by the developing corroded depth on the periphery of pipe as an index. Judgment was conducted in accordance with the standards speculated in "Rehabilitation Guidelines on Water Supply Facilities (May 2005)" published by Japan Water Works Association. Photo-1 shows the conditions that corroded depth on DIP reached 1.2mm (left: installed in 1973) and 1.4mm (right: installed in 1979) respectively.

Photo-1 Corroded Conditions on DIP

4-3 Deteriorated Degree Rating by Study Results

Deteriorated degree on each pipeline was respectively set up by forecasting the corroded depth on the periphery of pipe from the point view of causticity of underground soil and the years buried based on the results of field study.

Among of the pipelines installed by DIP and CIP, the pipeline lengths were respectively forecast as follows:-

i) 0km for Rank 1 defined on condition that breakthrough corroded;
ii) 2.7kms for Rank 2 defined on condition that corrosion was being developed and cannot resist the internal/external pressure;
iii) 25.9kms for Rank 3 (require planning on rehabilitation schedule) defined on condition that corrosion was being developed and insufficient safety factor against the internal/external pressure;
iv) 600kms for Rank 4 (require 2nd diagnosis within 10 years) defined on condition that corroded depth was over 2mm; and
v) 73.5kms for Rank 5 (require 2nd diagnosis within 20 years) defined on condition that corroded depth was below 2mm.

With regard to the block distribution system, it is absolutely necessary to conduct the rating on the whole pipe types since where other type pipes such as VP etc. already exist. Therefore, rank by average deteriorated degree on block distribution system was respectively estimated by applying the correlation with the deteriorated degree rank on DIP and CIP using the accident rate (case/km) under normal conditions.

4-4 Deteriorated Degree Rating by Figures

Rating grade on deteriorated degree was thus set up by the geometric mean of three items: i) deteriorated degree rank; ii) accident rate under normal conditions; and iii) installation elapsed years. With regard to deterioration degree rank, deteriorated degree over Rank 4 without any deterioration developing was set as the highest grade (100 points) and Rank 2 below rehabilitation is required since the deterioration was proceeding as the lowest grade (1 point). With regard to average accident rate, 0.003 case/km was set as the highest grade (100 points) and below 0.4 case/km was set as the lowest grade (1 point). With regard to the installation elapsed years, below forty (40) years of legal useful life as the highest grade (100 points) and over fifty-five (55) years was set as the lowest grade (1 point). These three (3) items were estimated between the highest grade and the lowest grade by linear equation. Figure-6 illustrates the evaluation results on block distribution systems within Hachinohe area.

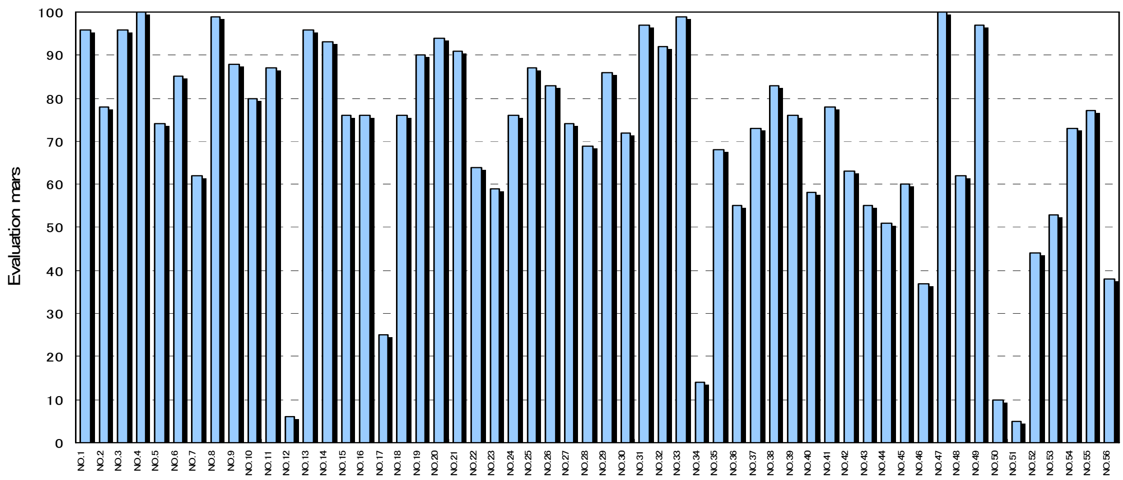


Figure-6 Deteriorated Degree Rating
(Block Distribution Systems within Hachinohe Area)

5. SEISMIC RESISTIVITY RATIO ON WATER SUPPLY ROUTE TOWARD CRITICAL SITE FACILITIES

Presently, there are 472 hospitals and 399 schools to be established as critical site facilities by mapping system. Pipelines corresponding to major water supply routes toward these critical sites have consequential level of importance in comparison with the same of other pipelines. Accordingly, pipe type and total length of those pipelines set as the water supply route toward the critical sites were respectively summarized by pursuing the hydraulic analysis on flow rate and flow direction for the water supply route from the distribution reservoir. As a result, seismic resistivity ratio (quake-resistant pipeline length over total length of water supply route pipeline) represents such water supply route was estimated. Table-4 shows that seismic resistivity ratio by three types was nearly 30%.

Table-4 Seismic Resistivity Ratio on Water Supply Route toward Critical Site Facilities

Target Facilities	Pipe Length of VP (km)	Pipeline Total Length (km)	Quake-resistant Pipeline Length (km)	Seismic Resistivity Ratio (%)
Hospital	15.1	413.0	131.9	31.9
School	24.6	493.8	151.7	30.7
School & Hospital	38.5	652.5	197.4	30.3

6. PROBLEMS TO BE IMPROVED

ACP and CIP were replaced by nationally-subsidized project since 1997 after the victim of Sanriku Harukaoki Earthquake. Therefore, these pipes steadily face to a relief. However, rehabilitation on VP which shows high damage rate was not being progressed in comparison with the same of ACP and CIP. It was proved that numerous damages on pipeline with damage counts of 315 cases occurred. Consequently, selective rehabilitation by quake-resistant pipes is required in the future.

Thirty-seven (37) days necessary for recovery were forecast; however, it is understood by restoration simulation that pipelines cannot be reconstructed within twenty-eight (28) days as targeted by Hachinohe Regional Water Supply Authority. Thirty-three (33) restoration work teams are required to clear such goal. It can be considered that seismicity will be improved by the rehabilitation on VP in the future. However, it is necessary to review the well-developed backup and assistance system in cooperation with the pipe construction interests or adjacent water suppliers to aim at earlier restoration.

With regard to existing block distribution systems, there exist various scales on number of customers such maximum 12,300 households as the first on the list. It is necessary to lessen the extent of impact by water suspension in case of earthquake disaster and shorten the recovery period by further subdividing those block distribution systems with numerous numbers of customers.

With regard to deteriorated degree rating, the tendency on deterioration has not been to progress by the underground surroundings to the existing pipelines. However, there exists a risk that equivalent water service at the moment may not be sustained in the future caused by the sharp increase on creaky pipes have a high proportion of pipeline network if the scheduled rehabilitation on DIP were not put into practice.

Seismic resistivity ratio on pipelines which supply water to the important facilities such as hospital or school was not satisfactory with about 30% only. In addition, there exists CIP with large-size caliber which seismic resistivity was not applied yet. Water service up to such critical sites may become impossible in case of earthquake occurrence if such water supply routes were still not reinforced with seismic resistivity. Especially, groundwater use may be impossible in case of earthquake disaster since polyclinic hospital as a key disaster-prevention facility possessed private water supply by groundwater. Therefore, it is indispensible to assign the highest priority to aim at 100% of seismic resistivity ratio on the water supply pipelines up to such hospital acted as a disaster-prevention facility.

7. CONCLUSION

Rating by quantitative figures on the effect resulted from the quake-resistant pipeline project h being conducted so far may act as an easy-to-understand index for both municipalities or consumers and water suppliers. This time, 143 block distribution systems formulated pipeline network were respectively rated and present condition (seismicity and deteriorated degree) for each block distribution system was thus grasped.

We are in an age when those water supply pipelines installed before 1970 must be selectively rehabilitated in the hereafter. However, such quake-resistant pipeline project to be intensively implemented during short period may face financial difficulties taking the present condition with the reduction on population served and water demand into consideration.

Therefore, effective and efficient pipeline rehabilitation project must be conducted by verifying the project progress on measures against earthquake. It is important that such progress must be continuously managed in conjunction with the setting of future numerical targets by taking vantage of the index resulted from this figure in the future.

REFERENCES

[1] Aomori Prefecture: The Study Report on Damage Forecast by Earthquake and Tsunami in Aomori Prefecture (March 1997)

[2] Japan Water Works Association: Earthquake Damage Forecast on Water Supply Pipelines (November 1998)

[3] Japan Water Works Association: Rehabilitation Guidelines on Water Supply Facilities (May 2005)

[4] Japan Water Research Center: Guidelines on Diagnostic Function of Water Supply Facilities (April 2005)

Hydraulic Performance Analysis of Water Systems under Major Earthquakes

Gee-Yu Liu[1] and Hsiang-Yuan Hung[2]

ABSTRACT

The assessment of seismic performance of water systems is very important to urban earthquake disaster preparedness and mitigation. In this research work, a technology for such implementation has been developed based on earthquake scenario simulation and hydraulic analysis of pressured pipe flow. The water system of Taipei Water Department (TWD) was selected as a test bed for case study. Its serviceability following two most likely major earthquakes around Taipei metropolitan area has been quantified.

INTRODUCTION

Service disruption of water system following earthquakes may cause serious inconvenience to the daily life of people in disastrous areas. Medical caring, sanitation, fire-fighting and so forth may be seriously affected, too. Moreover, the time needed for the recovery of water supply is usually much longer than that for, says, electricity or telecommunication due to the difficulties raised by the damage in buried water pipelines. It is highly desirable to facilitate water utility managers with a seismic scenario simulation tool for estimating the likely service disruption following earthquakes. Measures could be taken then to improve the seismic preparedness and emergency response more appropriately. One that best exemplifies this is the software called GIRAFFE (Graphical Iterative Response Analysis for Flow Following Earth quakes) developed in the Cornell University (Shi 2006, Wang 2006). Since 2006, LADWP (the Los Angeles Department of Water and Power) has formally adopted GIRAFFE as a decision-supporting tool for its water system (MCEER, 2006).

In this research work, a technology for assessing the seismic performance of water systems has been developed based on earthquake scenario simulation and pipe flow hydraulic analysis. In the following sections, key elements in this technology, namely the models of strong motion attenuation and pipe repair rate, the pipe damage location simulation and the pipe damage model, will be introduced. The step-by step procedure for water system seismic assessment will be explained. The water system of Taipei Water Department (TWD) will be selected as a test bed for case study. Its serviceability following two most likely major earthquakes around Taipei metropolitan area will be quantified.

[1] Associate Research Fellow, National Center for Research on Earthquake Engineering, Taiwan.
[2] Assistant Researcher, National Center for Research on Earthquake Engineering, Taiwan.

ELEMENTS OF SEISMIC SCENARIO SIMULATION OF WATER PIPE DAMAGE

Strong Motion

The intensity of earthquake-induced ground motion can usually be predicted by using appropriate attenuation laws. For the case of Taiwan region, a formula for peak ground acceleration (PGA) has been proposed (Jean et al. 2002), which reads

$$\text{PGA} = 0.003694 \cdot e^{1.7538 \cdot M} \cdot [R + 0.1222 \cdot e^{0.7832 \cdot M}]^{-2.0564} \tag{1}$$

where M and R are the earthquake magnitude and the distance (in Km) from the site to the seismic source, respectively. This formula is of Campbell form and the coefficients were determined from the ground motions of 15 past earthquakes in Taiwan. In addition, a standard deviation of 0.68-0.75 (in natural logarithmic scale) of the ground motion data with respect to the formula was reported. Figure 1 depicts the attenuation curves of this formula at selected earthquake magnitudes. Since the PGA value (in g) by Eq. (1) is for a hard site, the effect of soil amplification has to be considered according to site classification.

Pipe Repair Rate

Repair rate (*RR*) is defined as the number of repairs (or damages) per unit pipe length (km throughout this paper). It is widely employed to indicate pipe fragility. Numerous investigations have been made to formulate the relationship between pipe repair rate and earthquake-induced strong motions and ground failures. The pipe materials and diameter affect the pipe repair rate, too. Particularly, an approach has been proposed to achieve the repair rates caused by ground shaking and by ground deformation separately (Yeh et al. 2006). Following this approach, a regression model for the effect of ground shaking (PGA in gal) solely can be expressed by

$$RR = 1.028 \times 10^{-3} \cdot \text{PGA}^{0.9735} \qquad (\text{R}^2 = 0.9388) \tag{2}$$

Here, the six empirical data points were attain from the ground deformation-free areas in Taiwan in the 1999 Chi-Chi earthquake. Pipes with a diameter equal to or greater than 200mm were selected for use, which consist of 596.13km of pipe length and 158 repairs, yielding an average of 0.265 repairs per km (cf. 0.439 repairs per km to pipe mains in the 1995 Kobe earthquake (ALA 2001), probably combined with the effect of ground deformation).

The empirical evidence has strongly indicated that large pipes over 12-inch (300mm) diameter have lower repair rates than do common diameter distribution pipes of 4 to 12-inch diameter (ALA 2001). As a result, judgment has been made here to revise Eq. (2) for the implementation to large pipes. The following equation was tentatively proposed in this study to simulate pipe damages:

$$RR = \begin{cases} 1.2 \times 10^{-3} \cdot PGA^{\alpha} & \phi \le 300\text{mm} \\ 0.8 \times 10^{-3} \cdot PGA^{\alpha} & 300\text{mm} < \phi \le 500\text{mm} \\ 0.4 \times 10^{-3} \cdot PGA^{\alpha} & 500\text{mm} < \phi \end{cases} \tag{3}$$

where $\alpha = 0.9735$ .Various curves of pipe repair rate have been illustrated in Figure 2.

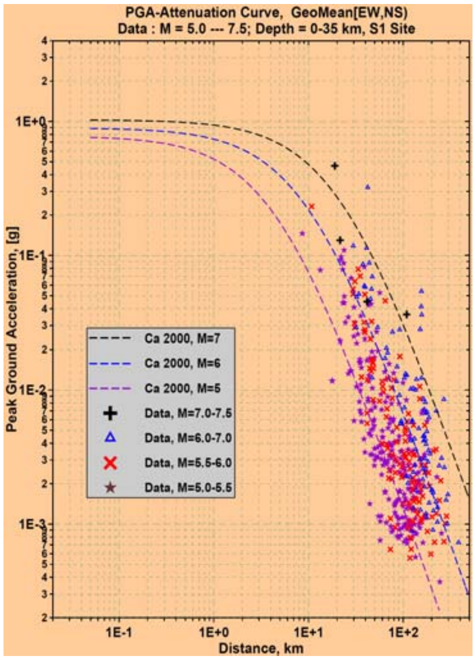


Figure 1. The attenuation curves for Taiwan region at selected earthquake magnitudes; source: (Jean et al. 2002).

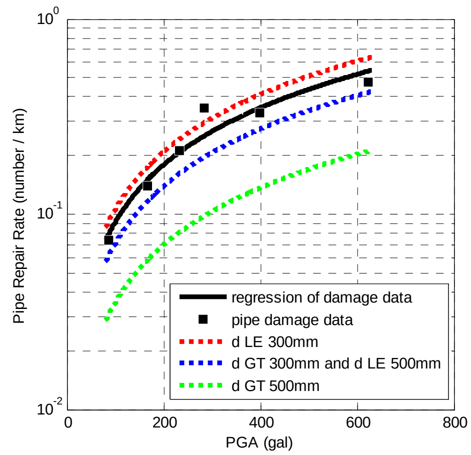


Figure 2. Various curves of pipe repair rate.

Pipe Damage Location

To generate the locations of pipe damage probabilistically, it is conventionally assumed that pipe damage follows a Poisson process with a mean arrival rate equal to repair rate. The locations can be generated pipe-by-pipe then by assuming that each pipe has a fixed value of repair rate (Shi 2006, Wang 2006). However, for a wide spreading pipe network, it is likely that there exist long pipes with varying repair rate. This conventional approach won't work in such case unless otherwise modified. Also, it lacks control over the total number of generated pipe damages (and locations). While various factors (e.g. the uncertainty in ground motion or pipe repair rate) are under investigation, the lack of control over the total number of generated pipe damages may lead to confusion in identifying the effect of these factors.

In this study, a new approach based on the expected number of damages of each pipe is proposed as follows. Suppose there are a total of N pipes in a pipe network. Each pipe has a length of ℓ_i ($i = 1,\ldots, N$). Let the unit length of a pipe segment be L. Define x_{ij} as the mid point of the j-th segment of the i-th pipe ($j = 1,\ldots, J_i$; the length of the last segment L_{iJ_i} may be less than L). Then, the expected number of damages in the pipe segment at x_{ij} can be expressed as

$$e_{ij} = \begin{cases} RR\left[\mathrm{PGA}(\mathrm{x}_{ij})\right]\cdot L & j = 1,\ldots, J_i - 1 \\ RR\left[\mathrm{PGA}(\mathrm{x}_{iJ_i})\right]\cdot L_{iJ_i} & j = J_i \end{cases} \tag{4}$$

where $\mathrm{PGA}(\mathrm{x}_{ij})$ is the value of PGA at x_{ij}. From Eq. (4), the expected number of total damages of the pipe network is

$$E_R = \sum_{i=1}^{N} \sum_{j=1}^{J_i} e_{ij} \tag{5}$$

which is the accumulated expected number of damages of all pipe segments. If an interval of real number $[0, E_R]$ can be divided sequentially into shorter real-number segments of length e_{11}, e_{12}, …, e_{1J_1}, e_{21}, e_{22}, …, e_{NJ_N}, then there exists a one-on-one mapping relationship between a specific real-number segment and a specific pipe segment. To simulate a damage location, a number is first selected by a random number generator following the uniform distribution between 0 and 1. It is then scaled by multiplying E_R and the resultant number is used to locate the corresponding real-number segment, says e_{nm}. Finally, the mid point x_{nm} of the *m*-th segment of the *n*-th pipe is designated as a pipe damage location. The same process can be conducted for E_R times to designate all the pipe damage locations.

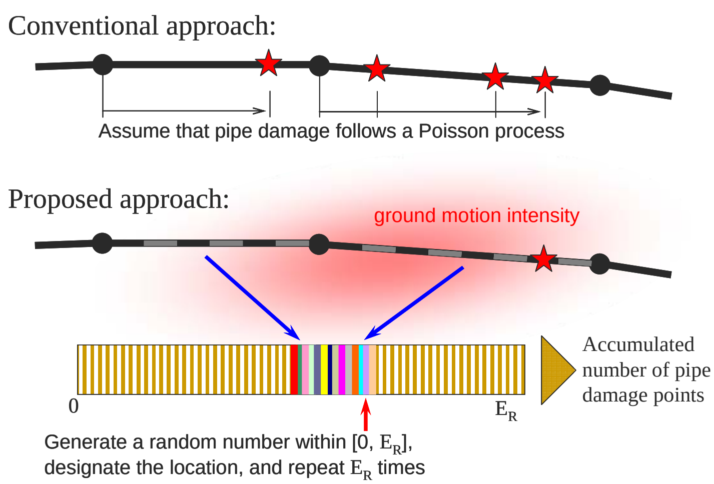


Figure 2. The comparison of two approaches for deciding the locations of pipe damage.

A comparison of the two approaches for deciding the locations of pipe damage are depicted as the schematic diagrams in Figure 2. The new approach possesses several merits. Firstly, it allows a varying repair rate along a pipe, which has been divided into shorter pipe segments in advance. Secondly, the probability a pipe segment is designated as being damaged is rigorously consistent with its expected number of damages. Finally, the number of generated locations is guaranteed as the same value of E_R, the expected number of damages.

It is worth mentioning that, since only 20% of the total repairs of welded steel pipes are leaks and will affect the ability of a pipe to convey water, the final number of damage points needed to be dealt with in the hydraulic analysis is sometimes less then the expected number of damages.

Pipe Damage Model

A pipe break and its hydraulic model could be depicted as the schematic diagrams at the left side of Figure 4. At each of the broken ends, a reservoir and a short pipe with a check valve are needed being added to mimic the mechanism for water flowing into the atmosphere. To take into account the effect of a pipe break in simulation, several steps for modifying the hydraulic model of the associated water system have to be carried out. They are: (1) Decide the location and elevation of pipe break point, (2) Remove the original link (pipe segment), (3) Add two new nodes A and B at the location of pipe break point, (4) Add two new links connecting the original pipe segment ends to A and B, respectively, (5) Add two new nodes A' and B' with the elevation of pipe break point and designate them as reservoirs, and finally (6) Add two new links connecting A-A' and B-B' and specify them with one-way check valves, respectively.

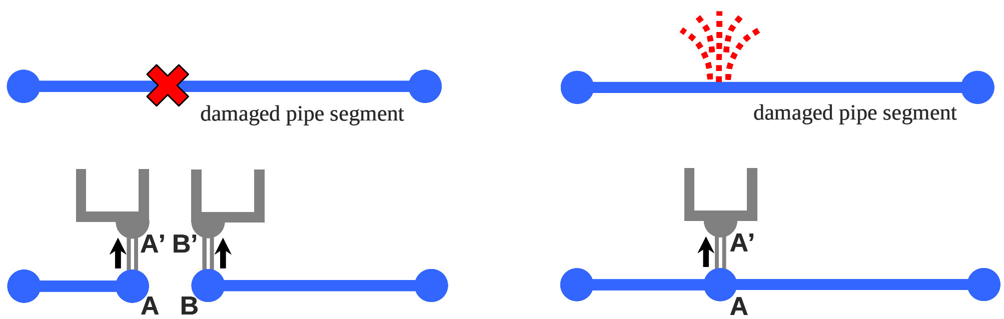


Figure 4. The pipe damage and hydraulic model for a pipe break (left) and a pipe leak (right).

On the other hand, a pipe leak and its hydraulic model could be depicted as the schematic diagrams at the right side of Figure 4. A pipe leak is hydraulically equivalent to a sprinkler with a specific discharge coefficient and an orifice size. This sprinkler is further proven to be equivalent to a fictitious pipe linking the original pipe and an added reservoir (Shi 2006, Wang 2006). A check valve is designated to the fictitious pipe to ensure that water flows from the leaking pipe to the reservoir. The corresponding steps for modifying the hydraulic model of the associated water system could be summarized as: (1) Decide the location and elevation of pipe leak point, (2) Remove the original link (pipe segment), (3) Add a new node A at the location of pipe leak point, (4) Add two new links connecting the original pipe segment ends to A, (5) Add a new node A' with the elevation of pipe leak point and designate it as a reservoir, and finally (6) Add a new link connecting A and A' and specify it as a fictitious pipe with a diameter of corresponding pipe leak model, and also specify it with a one-way check valve.

Table 1. GIRAFFE damage probability model for various pipe materials; source: (Shi et al. 2006).

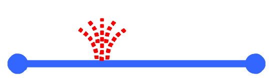
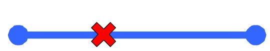
Damage type		Cast iron	Ductile iron	Riveted steel	Welded steel	Jointed concrete
Leak		0.8	0.8	0.8	0.2	0.8
Break		0.2	0.2	0.2	0.0	0.2

Table 2. GIRAFFE pipe leak probability model for various pipe materials; source: (Shi et al. 2006).

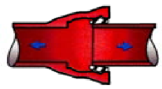
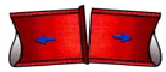
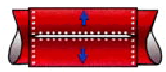
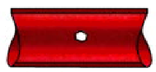
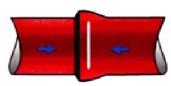
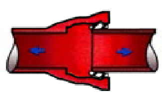
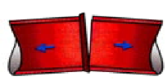
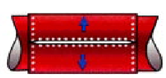
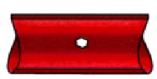
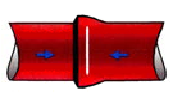
Pipe leak model		Cast iron	Ductile iron	Riveted steel	Welded steel	Jointed concrete
Annular disengagement		0.3	0.8	0.6	-	1.0
Round crack		0.5	-	-	-	-
Longitudinal Crack		0.1	0.1	0.3	-	-
Local loss of pipe wall		0.1	0.1	0.1	-	-
Local tear of pipe wall at welded slip joint		-	-	-	1.0	-

Table 3. GIRAFFE fictitious pipe diameters for pipe leak modeling; source: (Shi et al. 2006).

Pipe leak model		Diameter of the fictitious pipe	Values of parameters
Annular disengagement		$2\sqrt{tkD}$	$k = 0.3 \quad t = 10\text{mm}$
Round crack		$\sqrt{2\theta D}$	$\theta = 0.5^\circ = 0.00873$
Longitudinal Crack		$2\sqrt{LD\theta / \pi}$	$\theta = 0.1^\circ = 0.00175$
Local loss of pipe wall		$2\sqrt{k_1 k_2 D}$	$k_1 = k_2 = 0.05$
Local tear of pipe wall at welded slip joint		$2\sqrt{kWD}$	$k = 0.3 \quad W = 12\text{mm}$

In this study, the pipe damage model proposed by Shi and Wang (Shi 2006, Wang 2006) , termed as GIRAFFE model, was adopted. Following their findings from water pipelines and their damages in past earthquakes in the U.S., water pipe materials could be classified into 5 types: cast iron, ductile iron, jointed concrete, riveted steel and welded steel. Table 1 summarizes the occurrence probability of a leak or a break for damage in each pipe material. For all pipe materials except welded steel, the total number of repairs (damages) consists of 80% of pipe leaks and 20% of pipe breaks, respectively. While for welded steel pipes, only 20% of the total repairs are pipe leaks, and the rest damages are merely deformation and won't affect the pipe's ability to carry water. Furthermore, there are five different types of pipe leaks identified, namely the annular disengagement, round crack, longitudinal crack, local loss of pipe wall, and local tear of pipe wall at welded slip joint. Table 2 summarizes the occurrence probability of each type of pipe leaks, while Table 3 summaries the proposed diameter and geometrical parameters for the corresponding fictitious pipe employed in hydraulic modeling.

SEISMIC ASSESSMENT PROCEDURE OF WATER SYSTEMS

The procedure for assessing the seismic performance of water systems is illustrated in the flow chart in Figure 5. It reads

1. Read the input file for hydraulic analysis of the interested water network system. This file is usually prepared by the water utilities and is compatible with the employed analysis software in terms of data formatting. All attributes of the components in the water system (e.g. reservoirs, tanks, pumps, nodes and pipes) are defined in the file.
2. Simulate the pipe damages based on an earthquake scenario. A preprocessor has been specially developed in this study for modifying the input file as required by the above-mentioned earthquake scenario simulation of water pipe damage.
3. Check the connectivity of all nodes to the network system with simulated pipe damage. Remove the disconnected nodes by further modifying the input file.
4. Perform hydraulic analysis using EPANET.
5. Check the pressure at all nodes from the result of Step 4 and summarize the water supply by eliminating the demands at nodes of negative pressure.

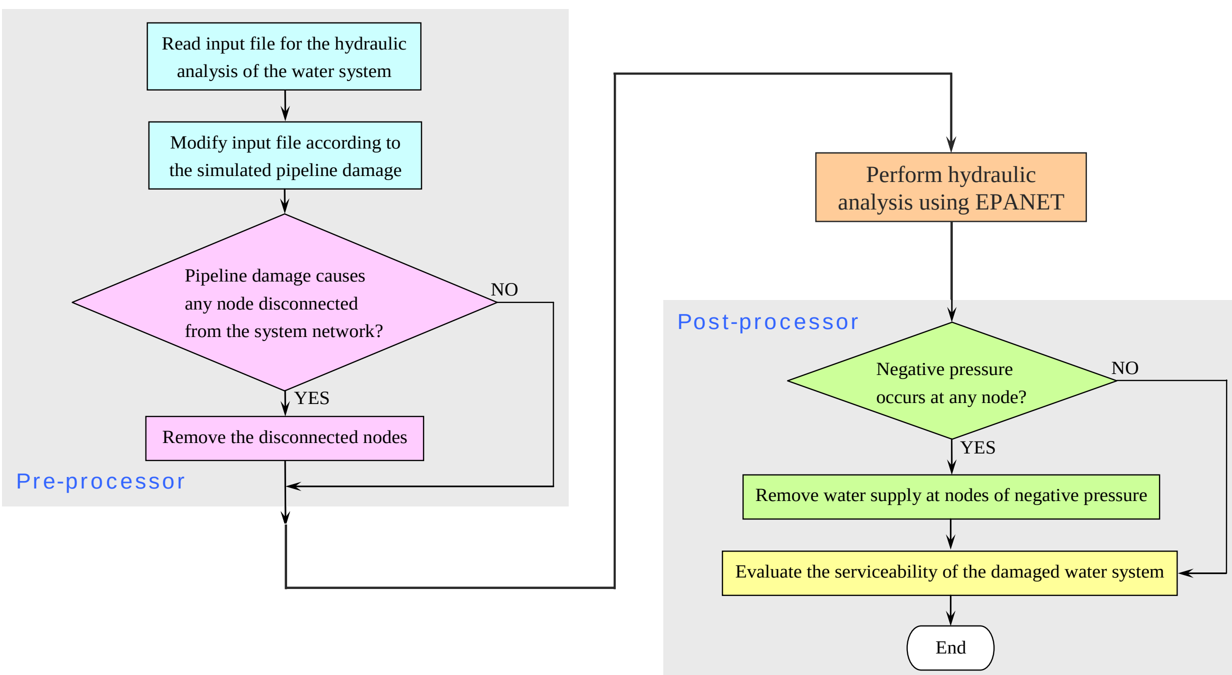


Figure 5. The flowchart for assessing the seismic performance of a water system.

In Step 4, the software EPANET is a computer program developed and maintained by the U.S. Environmental Protection Agency for the simulation of hydraulic and water quality behavior within a pressurized pipe network (Rossman 2000). It is free software and has been widely adopted by commercial packages, e.g. WaterCAD and MIKE NET, as their hydraulic engine. GIRAFFE employs the computer codes of EPANET for hydraulic analysis, too.

While performing hydraulic analysis of a water network with pipe damage, it is likely to predict negative pressure at some nodes. Negative pressure is generated due to the employment of the assumption that the pipe flows are always full and pressurized. However, water pipelines are not air-tight, especially when they are damaged, and the assumption no longer holds. Accordingly, hydraulic analysis of a damaged water network tends toward overestimating its ability to convey water. Elimination of negative pressure under such circumstance is advised. In this study, the approach proposed by Ballantyne et al. (1990), which assumes that no water will flow through negative pressure nodes, was employed in Step 5 of the assessment procedure.

CASE STUDY

The water system of the Taipei metropolitan area has been selected as a test bed for case study. This water system is operated by the Taipei Water Department (TWD). It provides service to the Taipei City as well as four other cities of the Taipei County (i.e., Shan-chung, Chung-ho, Yeong-ho and Hsin-dian). TWD has a service region of 434 square kilometer separated into 10 service areas, and serves water to 1.51 million customers or 3.85 million people. The daily water supply is around 2.5 million tons. Table 4 summarizes the statistics of each service area including the numbers of nodes, pipes, pumps, tanks, reservoirs and total pipe length. The distribution of pipelines and 10 service areas of TWD are illustrated in Figure 6.

Table 4. Statistics of pipe networks of the TWD system.

Service Area		Nodes	Pipes	Pumps	Tanks	Reservoirs	Pipe Length (km)
1	Shih-lin Bei-tou	3,254	3,376	40	10	0	171.359
2	Shan-chung	2,289	2,366	18	3	0	102.684
3	Da-tung	4,288	4,421	27	2	0	143.565
4	Ming-sun	1,769	1,822	18	2	0	68.446
5	Nei-hu	2,591	2,673	10	2	0	116.349
6	Kung-kuan	3,691	3,796	17	2	0	142.553
7	Chang-hsin Nan-gang	4,985	5,127	32	3	1	193.524
8	Chunh-ho Yeong-ho	2,338	2,394	5	1	0	98.039
9	An-kang	587	601	12	2	0	28.829
10	Hsin-dian	2,716	2,799	20	3	0	130.933
Total		28,508	29,375	199	30	1	1,196.281

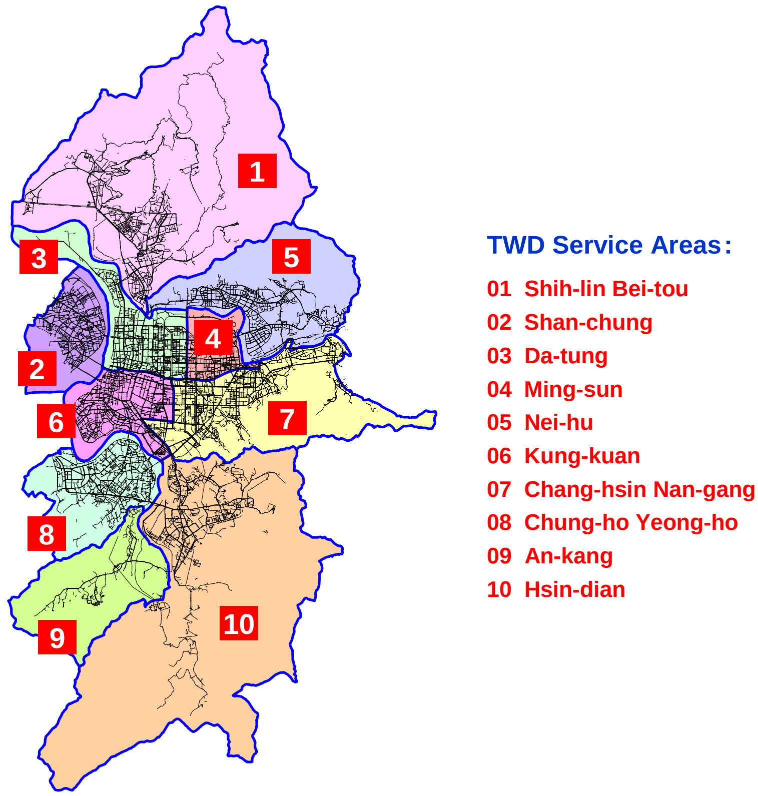


Figure 6. The distribution of pipelines and 10 service areas of the Taipei Water Department (TWD).

Two earthquake scenarios have been considered. They are the M5.9 earthquake associated with the Sanchiao fault and the M7.5 earthquake associated with the Hsincheng fault. They are among the most likely major earthquakes to occur in the neighborhood of Taipei metropolitan area triggered by active faults. For each earthquake scenario, the assessment procedure has been conducted 100 times for the pipe network of each service area. The simulated distributions of serviceability index for the TWD water system under these two earthquake scenarios have been depicted in Figures 7 and 8, respectively. Here, the serviceability index (SI), the ratio of flow at demand nodes before and after an earthquake, is used to quantify the water network seismic performance in each service area. In the M5.9 case, the SI values vary between 0.45 and 0.89. While in the M7.5 case, the SI values vary between 0.44 and 0.92.

SUMMARY

A technology for assessing the seismic performance of water systems has been developed. Key issues including models of strong motion attenuation and pipe repair rate, and the pipe damage model have explained. Particularly, a new approach for the simulation of pipe damage locations based on the expected number of damages of pipe segment has been proposed. The system of Taipei Water Department (TWD) was employed as a test bed for case study. Two most likely major earthquakes around Taipei metropolitan area have been considered. The simulated post-earthquake performance in terms of serviceability index (SI), the ratio of flow at demand nodes before and after an earthquake, has been identified for each service area and for each earthquake scenario.

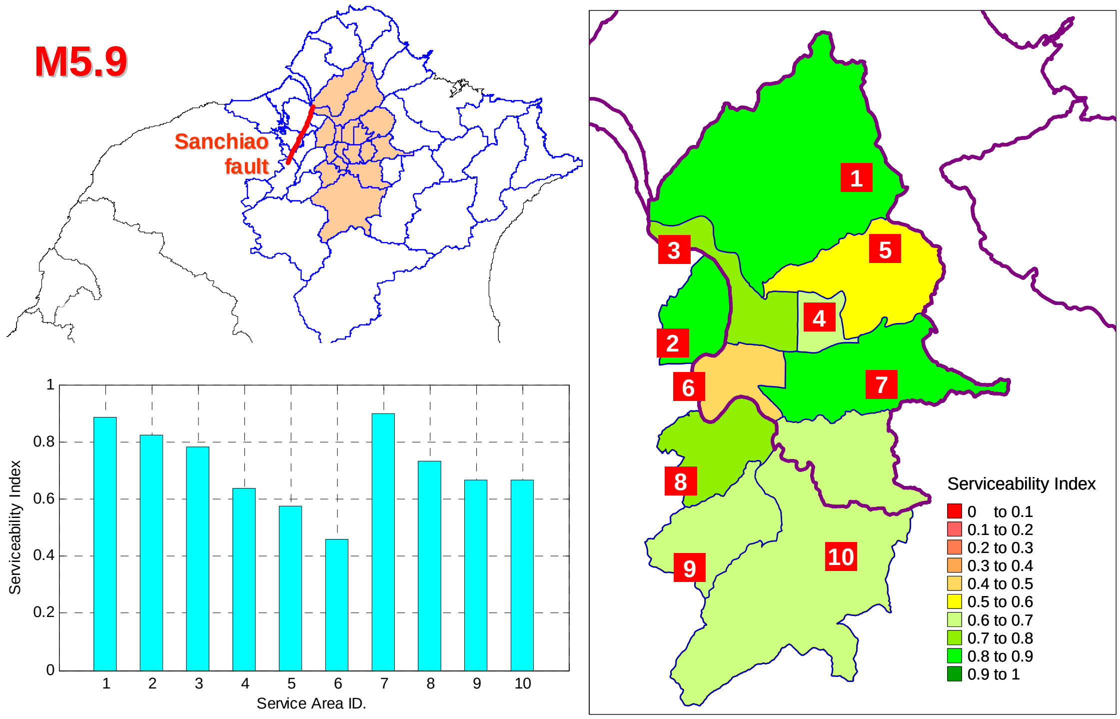


Figure 7. The simulated distribution of SI value for the TWD water system following the considered M5.9 earthquake associated with the Sanchiao active fault.

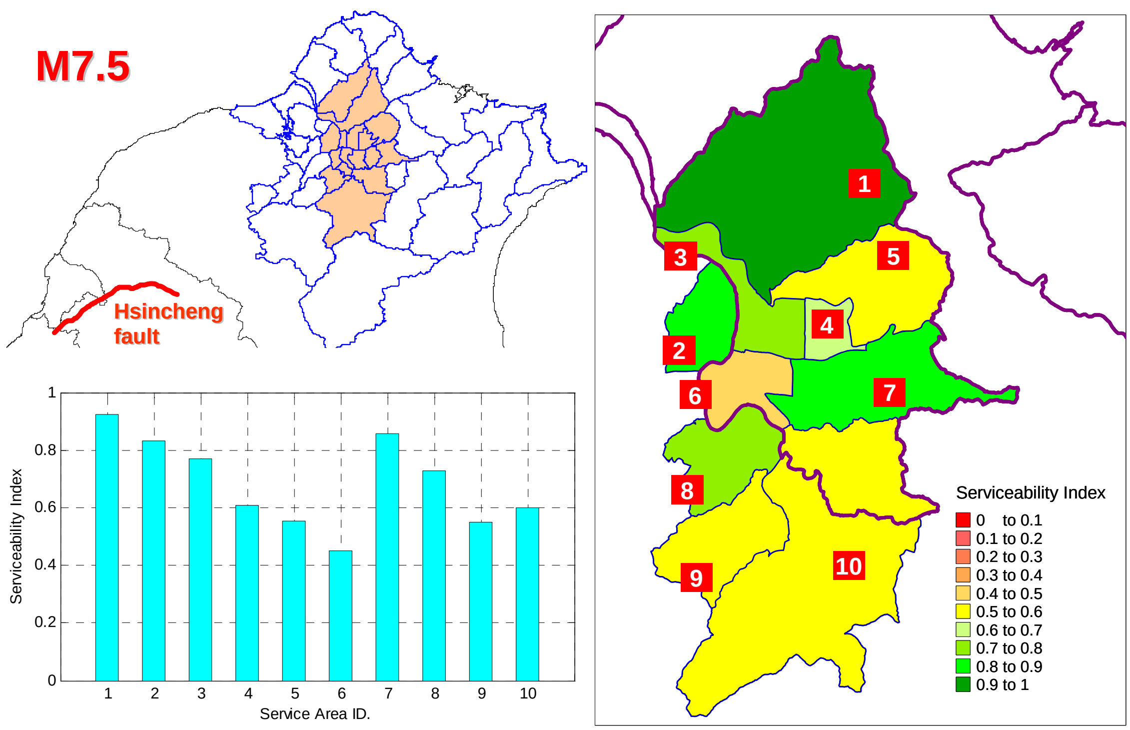


Figure 8. The simulated distribution of SI value for the TWD water system following the considered M7.5 earthquake associated with the Hsincheng active fault.

ACKNOWLEDGMENTS

This research work was funded by the National Science Council, Taiwan under Grant No. NSC-96-625-Z-492-007, and was carried out using the network hydraulic data of the Taipei Water Department. Both are gratefully acknowledged.

REFERENCES

[1] ALA (American Lifelines Alliance) 2001. Seismic Fragility Formulations for Water Systems, http://www.Americanlife linesalliance.org.
[2] Ballantyne et al. 1990. Earthquake Loss Estimation Modeling of the Seattle Water System, Technical Report, Kennedy/Jenks/Chilton, Federal Way, WA.
[3] Jean, W.-Y. 2005. Private communications.
[4] MCEER 2006. http://mceer.buffalo.edu/publications/bulletin/ 06/2002/03_ladwp.asp.
[5] Rossman, L. A. 2000. EPANET 2 User's Manual, EPA, Cincinnati, OH.
[6] Shi, P. 2006. Seismic Response Modeling of Water Supply Systems, Ph.D. Dissertation, Cornell University, Ithaca, NY.
[7] Shi, P., O'Rourke, T. D. and Wang, Y. 2006. Simulation of Earthquake Water Supply Performance, Proc. 8th U.S. Nat. Confer. Earthquake Eng., Paper No. 1295, San Francisco, CA.
[8] Wang, Y. 2006. Seismic Performance Evaluation of Water Supply Systems, Ph.D. Dissertation, Cornell University, Ithaca, NY.
[9] Yeh, C.-H., Shih, B.-J., Chang, C.-H., Chen, W., Liu, G.-Y. and Hung., H.-Y. 2006. Seismic Damage Assessment of Potable Water Pipelines, Proc. 4th Int. Confer. Earthquake Eng., Paper No. 247, Oct. 12-13, Taipei, Taiwan.

Simulation of Los Angeles Water Supply and Distribution in Response to M7.8 San Andreas Earthquake

Craig A. Davis, Natalia Romero, and Thomas D. O'Rourke

ABSTRACT

The Los Angeles Department of Water and Power (LADWP) is the largest municipal utility in the United States. Water supply for Los Angeles is obtained from three major aqueduct systems, the Los Angeles, California, and Colorado River Aqueducts, and local groundwater. Water is distributed to over 4 million people and businesses through a network of 11,691 km of trunk and distribution pipelines. This paper describes the potential impacts of a magnitude 7.8 San Andreas Fault earthquake scenario to the three major aqueducts providing domestic and industrial water supply to Los Angeles and the water transmission and distribution system. The simulation was performed as part of the 2008 ShakeOut earthquake preparedness drill, which was the largest exercise of its kind in US history. All three aqueducts cross the postulated fault rupture and suffer severe damage, resulting in an estimated 4 to 18 months restoration period. The water transmission and distribution system was numerically simulated using a decision support system for water supply response to earthquakes. The simulation covered all 11,691 km of pipelines and related facilities (e.g., tanks, reservoirs, pressure regulation stations, etc.) in the LADWP system, plus the performance of the Los Angeles Aqueducts. The main findings of the simulation are summarized, with serviceability quantified as the ratio of water flow after to water flow before the earthquake on both a system-wide level and for 15 different water service areas. The simulation accounts for the effects of seismic ground waves and permanent ground deformation associated with fault rupture, liquefaction, and landslides. Severe deterioration in water delivery is shown over 24 hrs due to damaged and leaking pipelines. The simulation shows nearly 2,700 locations of pipeline repair of which approximately 5% are breaks and 95% are leaks. The system serviceability is approximately 34% after 24 hrs, which means that 66% of the normal water demand cannot be met. Local emergency water supplies may not be sufficient to match the duration of aqueduct repairs, requiring severe water rationing. Restoration of water flows to all customers may take up to several months and restoration of pre-earthquake water demand may take a year. The earthquake scenario estimates a super conflagration in a densely populated area of Los Angeles, where water serviceability is no more than 31%, causing difficulties for firefighting. Business interruptions due to long term water rationing are estimated to have a dramatic impact on the regional economy. Results of this study are useful for (1) identifying how the LADWP and other local water supply agencies can prepare for such a large regional earthquake, and (2) developing and prioritizing pre-earthquake mitigation and post-earthquake restoration strategies.

Craig A. Davis, Waterworks Engineer, Geotechnical Engineering Group, Los Angeles Department of Water and Power, 111 N. Hope Street, Room 1368, Los Angles, CA, 90051; email: craig.davis@ladwp.com.
Natalia Romero, Teaching Assistant, Department of Civil and Environmental Engineering, Cornell University. Hollister Hall, Ithaca, NY 14853; email: nr229@cornell.edu.
Thomas D. O'Rourke, Professor, Department of Civil and Environmental Engineering, Cornell University. Hollister Hall, Ithaca, NY 14853; email: tdo1@cornell.edu

INTRODUCTION

A magnitude (M) 7.8 earthquake scenario on the southern San Andreas Fault was recently prepared [1] to gain a better understanding of how such an event may impact critical infrastructure, including water supply and distribution systems, and identify the physical, social, and economic consequences of a major earthquake in Southern California. This scenario provides a realistic description of possible fault displacement and shaking throughout Southern California, and is useful for performing more detailed evaluations of water supply impacts from a San Andreas Fault event. The purpose of this paper is to present an initial evaluation, using the information presented for the M7.8 San Andreas Fault scenario, to understand expected impacts to the Los Angeles Department of Water and Power (LADWP) water transmission and distribution network. Additionally, impacts to the Los Angeles Aqueducts (LAA), California Aqueduct, and Colorado River Aqueduct (CRA) and a timeframe for which they may realistically be expected to return to service are reviewed. Results of this study will identify areas where further investigation is needed regarding water system damage and restoration, and provide a better understanding of the risks posed to Los Angeles by the San Andreas Fault.

EARTHQUAKE SCENARIO

A M7.8 earthquake scenario on the southern San Andreas Fault was recently developed for use in the Great Southern California ShakeOut, the largest earthquake exercise ever undertaken in the United States. Hereafter, this scenario will be referred to as the ShakeOut Scenario. The scenario development was based on a plausible event and resulting impacts, not a worst case scenario, and accomplished through collaboration of many contributors coordinated by the United States Geologic Survey [1]. Figure 1 shows the Southern California region, ruptured portion of the San Andreas Fault, and shaking intensity for this scenario. The M7.8 earthquake has a hypothesized epicenter at the Salton Sea, near the southern most end of the San Andreas Fault, and ruptures 300 km north to Lake Hughes. The scenario event and resulting impacts described in this section are summarized from more detailed reports presented in [1]. Fault rupture displacements exceed 9m and large ground motions result, especially near the northern end of the rupture where directivity effects cause large velocity pulses. Intense shaking is experienced over a large area of Southern California affecting over 22 million people. The Los Angeles and Ventura sedimentary basins trap and amplify seismic waves leading to several minutes of shaking. Ground failures from landslides, lurching, liquefaction and lateral spreading result in many locations of the severely shaken areas. Earthquake induced shaking from the main shock causes significant damage and disruption throughout Southern California. Over a million buildings suffer significant structural and non-structural damage, some resulting from collapse. Fault displacements disrupt many critical lifelines including major highways, railroads, power transmission, oil and natural gas, fiber optic communication, and water supply crossing the fault, while shaking and other ground failures cause additional widespread damage to lifelines throughout Southern California. Transportation corridors and water supplies are severely disrupted for long periods of time. Hundreds of water systems, large and small, are damaged by the earthquake disrupting the supply and distribution to customers and the ability to fight fires. An estimated 1.5 million people in all of Southern California are without potable water immediately after the earthquake, and after 90 days over 180,000 are still without potable water due to distribution system damages. An estimated 1,600 ignitions require fire engine response, 1,200 resulting in large fires causing extensive damage. Several fires burn for over a week in highly urbanized and populated areas, some converging to make a super conflagration in the central Los Angeles basin.

The ShakeOut Scenario estimates approximately 50,000 injuries, 750 requiring special treatment, and approximately 1,800 deaths. Over half of the fatalities and special treatment victims result from fire. Over $210 billion in economic losses is estimated for this scenario earthquake, with slightly over half coming from property damage and the majority of remainder from business interruption. Large aftershocks, in the range of M6.4 to M7.7, will occur throughout the Southern California region causing additional damage, further compounding the disaster and increasing water supply and distribution impacts. Restoration and recovery will initially depend on the ability of local distribution systems to operate. Long term regional recovery will depend on the ability to restore imported water supplies.

SCENARIO IMPACTS TO WATER SUPPLY AQUEDUCTS

Figure 2 shows the three aqueducts providing over 70% of the annual average water supply for domestic and industrial use in Southern California: the LAA System [consisting of the First and Second Los Angeles Aqueducts (FLAA and SLAA, respectively)], owned and operated by the LADWP; the CRA owned and operated by the Metropolitan Water District of Southern California (MWD); and the California Aqueduct, owned and operated by the California Department of Water Resources, which was constructed as part of the California State Water Project (SWP). MWD is a state-created wholesale water agency, and a contractor to receive water from the SWP, who owns and operates a wholesale water supply system and provides the water to distribution agencies; MWD does not distribute water to industrial or domestic customers. The LADWP obtains 85% of the average annual water supply through these three aqueducts; the remaining 15% is from local groundwater sources. In addition, recycled water is used to supplement potable water for irrigation and industrial purposes, but is not considered a primary supply in the event of a disaster.

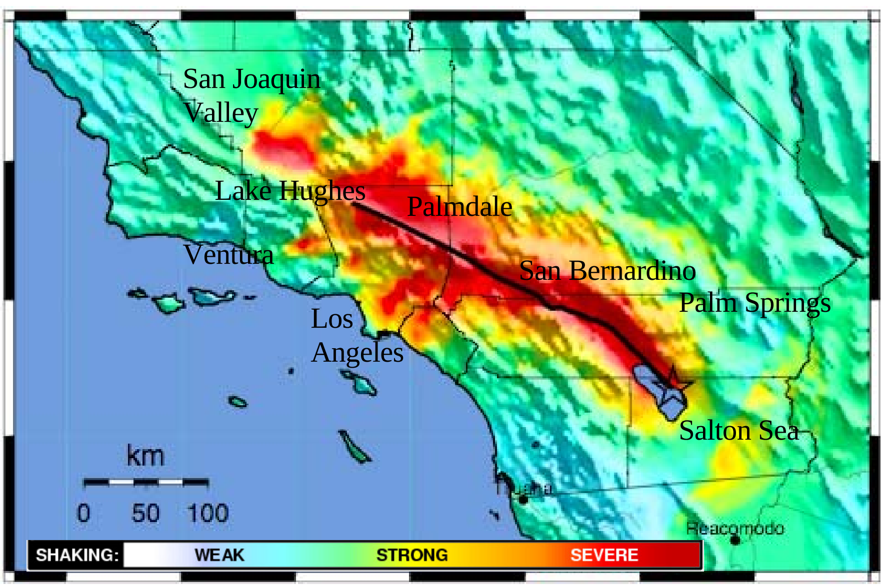


Figure 1. Southern California region showing the epicenter (star), fault rupture (heavy line), and shaking intensity for the M7.8 San Andreas Fault scenario (modified from [1]).

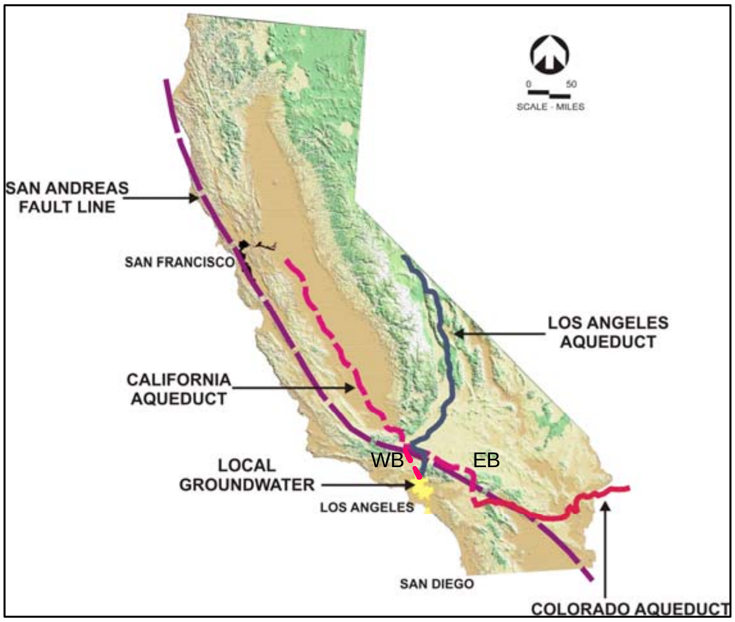


Figure 2. California map showing Los Angeles water supply sources and the San Andreas Fault. EB and WB are the California Aqueduct East and West Branches.

As shown in Figure 2, these three aqueducts cross the San Andreas Fault, and comparison with Figure 1 identifies they will be damaged by fault rupture displacements. Jones et. al. [1] identifies the aqueducts in Figure 2 will be damaged by fault displacements ranging from 0.04m to 4.9m. The largest movements occur on the San Andreas main trace, and smaller movements are distributed over other parallel faults and splays within the San Andreas Fault zone. At least 18 fault displacements offset the aqueducts; this estimate differs slightly from that provided by Jones et. al. [1, App. D]. The aqueducts are also affected by ground shaking and permanent ground deformations.

Damage sustained to these aqueducts result in a complete disruption of all imported water supplies for many months. Davis [2] performed a preliminary review of fault displacement, shaking, and other permanent ground deformation induced damage the aqueducts may sustain from the ShakeOut Scenario and developed the conceptual restoration schedules of Figure 3. As seen in Figure 3, a portion of the California Aqueduct may return to service within 4-months following this event, but the other aqueducts will not transport any water to Los Angeles for at least 1-year.

The LAA crosses the San Andreas Fault in the Elizabeth Tunnel. ShakeOut Scenario fault movements completely close off the tunnel. Additionally, the FLAA and SLAA sustain significant damage to channel, pipe, and tunnel sections on each side of the fault. Figure 3 shows repairs to Elizabeth Tunnel, taking at least 18-months, are critical to returning water flows.

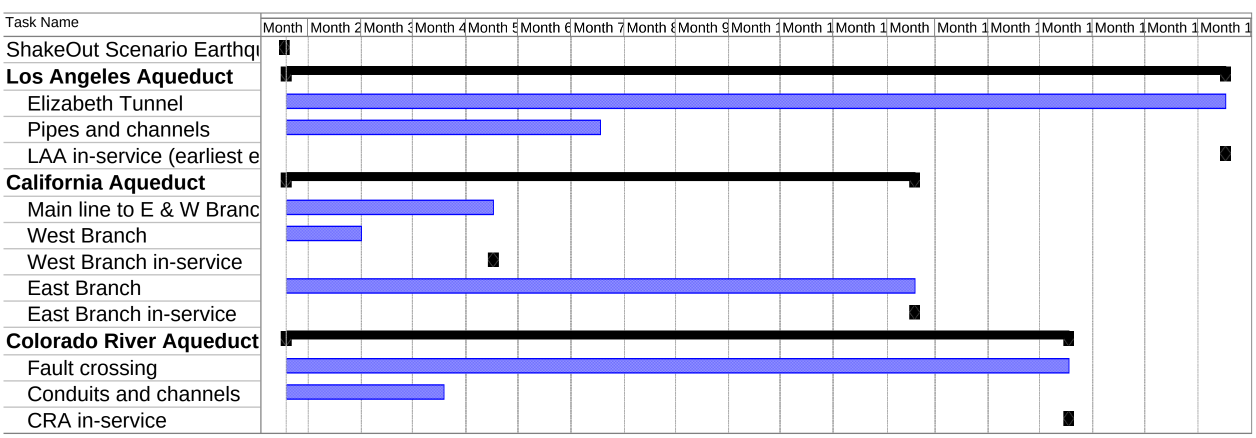


Figure 3. Aqueduct restoration conceptual time estimates [2].

North of Los Angeles, the California Aqueduct Main Line splits into the East Branch and West Branch. The fault rupture in Figure 1 stops south of the West Branch; only the East Branch suffers damage from the main shock fault rupture. The East Branch is estimated to be subjected to at least 13 to 17 ShakeOut Scenario fault offsets, with movements ranging from 0.04m up to 4.9m; these estimates are made by the author using data provided in [1]. Eight of the displacements will impose a combination of lateral and extensional movement in the channel embankments causing the aqueduct to pull apart by as much as 1.7m. This pull-apart movement will open gaps in the channel allowing soil to erode and large volumes of water to escape.

Approximately 300 km of the California Aqueduct will be shaken with strong to severe intensities. The East Branch channel crosses many zones of weak soil deposits and will suffer damaging deformations, resulting in several channel breaches, some possibly in urbanized areas. Pipes, tunnels, channels, power plants, and pumping stations of the Main Line and East Branch suffer some damage from shaking and permanent ground deformations. Figure 3 shows damage repairs on the Main Line are most critical to returning water deliveries through the West Branch. Figure 1 shows shaking of severe intensity in the southern San Joaquin Valley due to sediments amplifying seismic waves. The California Aqueduct Main Line channel is aligned along the historic Buena Vista Lakebed in this area and may experience large deformations and possible channel breach. Figure 3 shows the Main Line repairs taking up to 4-months and East Branch repairs may take at least1-year to complete.

The CRA crosses four fault strands making up the San Andreas Fault zone in the San Gorgonio Pass. The ShakeOut Scenario fault movements estimated to impact the CRA concrete conduit and tunnel sections range from 0.4m to 1.3m [1, App. E]. The fault slip in this region is complex and will include significant vertical components with the CRA upstream side lifted upward. Total uplift in the San Gorgonio Pass, which includes faulting and folding, may exceed 4m. This will reduce the CRA hydraulic capacity, which primarily flows unpressurized. The horizontal fault movement component will pull the CRA apart by several tenths of meters. In addition, over 140 km of the CRA will be subjected to strong to severe shaking damaging tunnels, conduits, channels, and pumping stations. Figure 3 shows repairs through the fault zone may take at least 15-months, mostly due to needed excavation to lower the CRA and re-establish the necessary hydraulic flow lines.

REGIONAL EMERGENCY STORAGE

An estimated 6 months of regional storage (surface and ground water) exists in Southern California for use following a great San Andreas Fault earthquake, if 25% average rationing is implemented [4]. Following the ShakeOut Scenario, groundwater pumping will significantly increase throughout the Southern California region. Figure 3 shows the LAA, California Aqueduct East Branch, and CRA restorations take much longer than 6 months. As a result, there does not seem to be enough emergency storage to sustain Southern California during the total aqueduct restoration duration for this plausible earthquake scenario. Restoration of the California Aqueduct West Branch within 6 months will help Los Angeles, but will not help all of the Southern California population due to limited ability to transmit West Branch water throughout the urban areas. Even those receiving West Branch water must continue rationing for many months to stretch this limited supply as far as possible. Inadequate local emergency storage will require increased rationing for longer durations and overdraft of local groundwater basins, leading to potential environmental problems. Evaluations are needed to identify ways to increase local storage and reduce aqueduct restoration time.

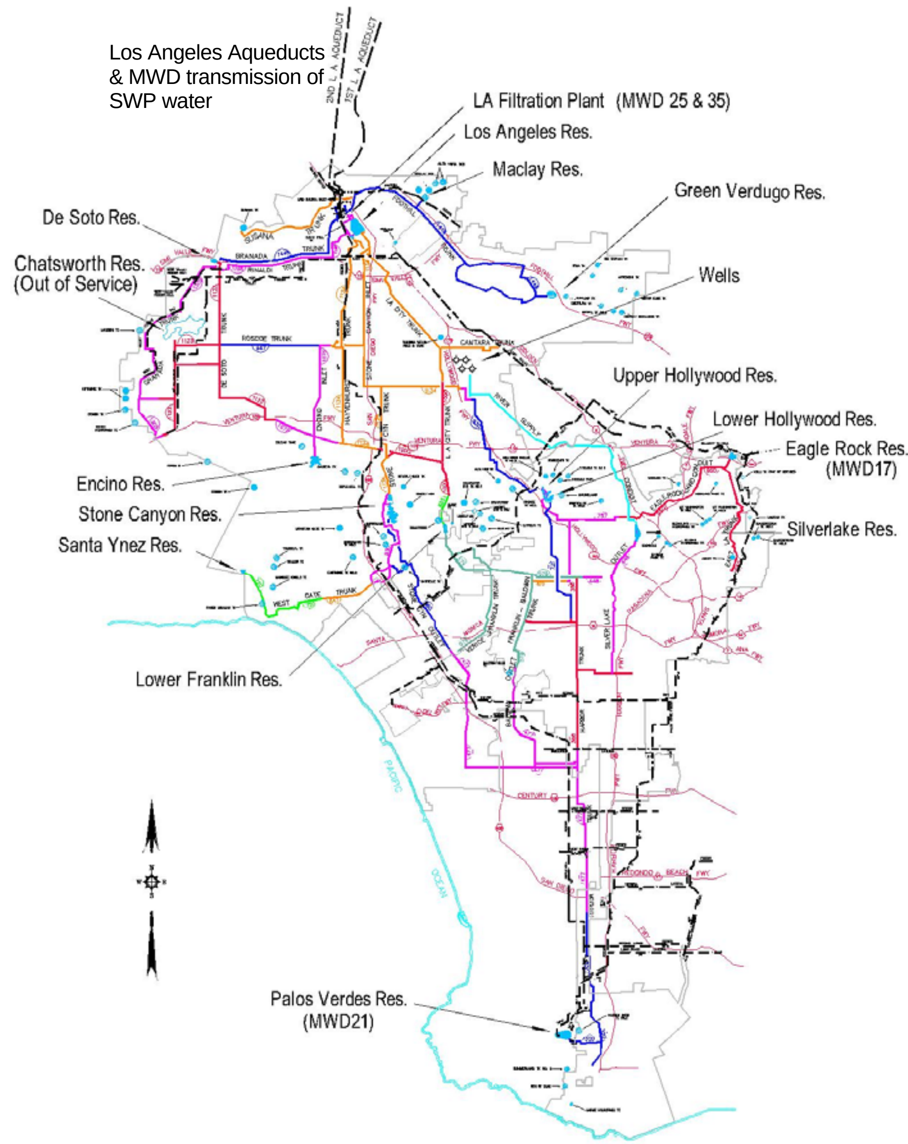


Figure 4. LADWP water supply and transmission system showing major trunk lines (solid bold lines), MWD supply lines (dashed lines), reservoirs (labeled), major tanks (solid circles), other facilities as labeled, and freeways (labeled solid lines), within the City boundaries.

LADWP WATER TRANSMISSION AND DISTRIBUTION

Figure 4 shows the LADWP water supply and transmission system within the City limits. The FLAA, SLAA, and MWD's transmission of SWP water enter the City from the north. MWD supplies CRA water at Eagle Rock Reservoir. The LADWP distributes water to over 4 million people within the City of Los Angeles covering a 1,204 km^2 area. This is accomplished using approximately 11,691 km of trunk and distribution pipelines ranging from 5 to 366 cm in diameter, 108 potable storage tanks and reservoirs having a total maximum capacity of 18.8 million m^3, 260 regulator stations, 80 pumping stations, three filtration plants, 25 chlorination stations, and over 712,000 service connections in 115 pressure zones. In addition, the LADWP maintains four raw water emergency storage reservoirs, having a maximum capacity of 30 million m^3, within the City. A recent survey of international water distribution agencies located in seismically active areas found the LADWP to have the largest volume of potable and non-potable emergency surface storage water [3]. Additionally, the LADWP is entitled to over 132 million m^3 of local groundwater per year. Further, LADWP has approximately 35 connections to MWD raw and treated water transmission lines throughout the City, providing additional supply points.

PIPELINE DAMAGE AND SYSTEM SERVICEABILITY

The City of Los Angeles is located approximately 50 km from the San Andreas Fault. Los Angeles is shaken for 55 seconds with strong ground motions reaching approximately 0.3g peak ground acceleration (PGA) and 200 cm/sec peak ground velocity (PGV)[1]. Romero et. al. [5, 6] shows the PGA and PGV spatial distributions in relation to the LADWP water system. The long shaking duration and large ground motions are a result of deep sedimentary basins underlying Los Angeles, which trap and amplify the propagating seismic waves. The PGA mostly effects surface structures; LADWP designs facilities for accelerations larger then 0.3g and therefore most should not suffer significant damage. The PGV motions cause exceptionally large transient ground strains resulting in significant overall pipe damage and permanent ground failure in weak soil zones. Pipes passing through ground failure zones suffer serious damage, resulting in large breaks.

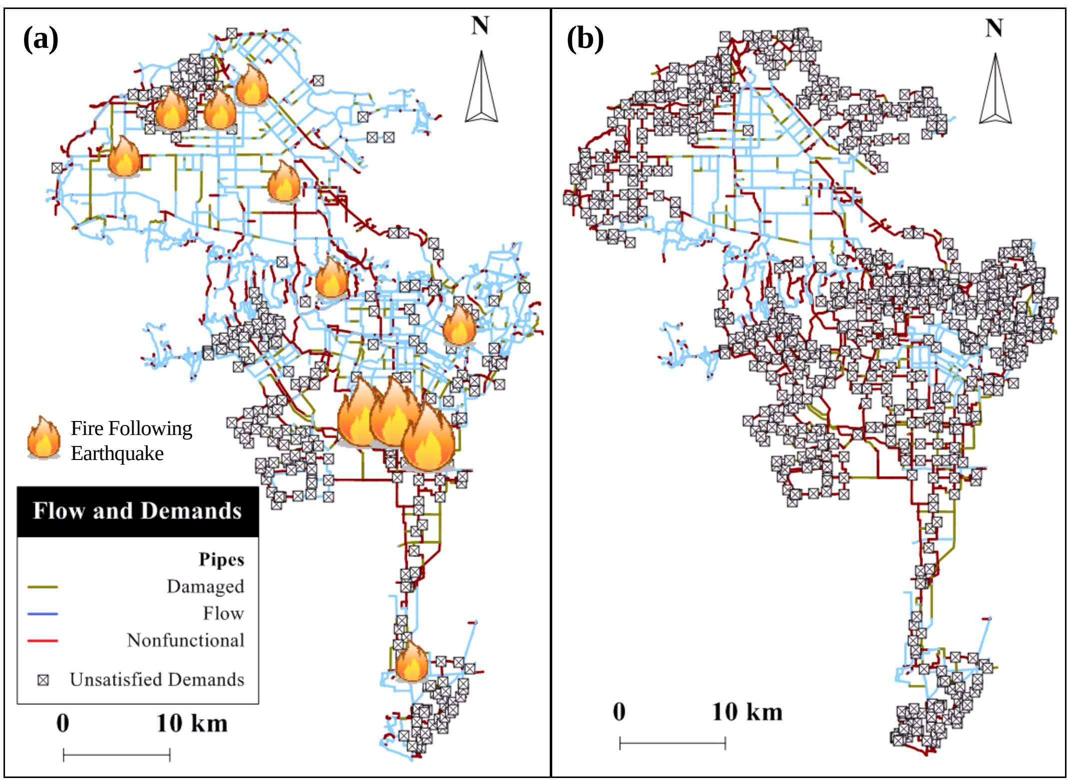


Figure 5. System flow state and unsatisfied demands for: (a) 0 and (b) 24 hours after the earthquake (modified from [5]). Predicted fire following earthquake locations, large fires and super conflagrations, are identified based on [7].

Romero et. al. [5, 6] simulated damage to the water pipeline network using a recently developed decision support system [8]. The system works in conjunction with EPANET [9] and a special program for damaged network flow modeling, known as Graphical Iterative Response Analysis for Flow Following Earthquakes (GIRAFFE). The simulation accounts for all 11,691 km of pipe and the effects of seismic ground waves and ground deformations associated with liquefaction, lurching, and oscillations. The simulation identified nearly 2,700 pipeline repair locations. Figure 5 presents simulation results at 0 and 24 hrs after the earthquake showing locations on the trunk line system where pipes are unpressurized and there is insufficient water flow to satisfy demand, prior to utilizing the raw water storage reservoirs. System serviceability, the ratio of water flow after to water flow before the earthquake, is approximately 76% immediately after the earthquake (at 0 hrs) and drops to 34% after 24 hrs. Severe deterioration in the ability to deliver water results over 24 hrs due to damaged and leaking pipelines. A 34% system serviceability means that 66% of the normal water demand, throughout the entire system, is not met one day after the earthquake. Some areas within the system will have higher or lower serviceability. The simulation results account for service line leakage and damage to interior piping of buildings, which draw more water from the system, but not for fire fighting demand. Leaking pipelines draw down tanks and reservoirs, causing some sections of the system to lose pressure and in some areas all local sources of stored water. Following such a large event, it will take approximately 24 hrs to mobilize the initial response to isolate and repair leaking pipelines. Thus, Figure 5b represents a likely flow state within one day following the earthquake, in the absence of fire demands.

WATER SERVICE RESTORATION

The time to restore a water system is dependent upon many factors, some of the most significant include the damage state, time to discover damage, the number of crews that report and when, travel times, available water supply, ability to isolate damaged pipes, and ability to use redundancy in the trunk line system. Restoring the Los Angeles water system after a ShakeOut Scenario event will be lengthy because of: a heavily damaged state, initial water supply is limited to the local storage, difficulties in identifying and reporting to damaged pipe locations because of inoperable communication and transportation systems, and limited crews reporting soon after the earthquake (if event does not occur during working hours).

Expected scenario damage to the pipe network is much more significant than to other water system structures; therefore, descriptions herein are limited to pipeline restorations. Following the event, a restoration scenario may proceed as follows. Initial focus will be on restoring water supply transmission through the trunk system and replenishing local storage with regional storage. Soon after the earthquake the damage severity will be recognized and a tap water purification notice will be issued City wide. Additionally, the State will issue a severe water conservation mandate to all of Southern California, effecting over 22 million people. Within 24 hrs, crews mobilize to begin isolating leaking pipes for repairs. Crews will be spread throughout the City, each focused in their own district and unable to support neighboring districts. The East Bay Municipal Utility District in Oakland, CA will initially mobilize mutual assistance within one day, providing aid to make repairs within two to three days. Identifying and prioritizing repairs is difficult due to communication and transportation difficulties, and because of widespread pipe damage throughout the system. There are about 150 estimated trunk line damage locations [5]. Many pressure zones are dry due to local storage draining out through leaking pipes, making pipe repairs difficult; damage locations are not easily identified without water in the pipes. Thus, managed water loss to identify pipe damage is a necessary part of system restoration. Groundwater pumping will be maximized, but the pumping capacity is insufficient to meet the emergency demand. The raw water emergency storage reservoirs will be utilized to pressurize the system for fighting fires and finding pipe damage. This will also reduce the loss of potable storage. Romero et al. [6] simulated the use of the raw water storage reservoirs and found that utilizing these reservoirs immediately after this large event significantly increases the system serviceability shown in Figure 5, which translates into more rapid system recovery.

Due to the large flows exiting damaged pipes and difficulties in isolating all the damage, most, if not all the, tanks and several large reservoirs will drain within days. System serviceability will continue to decline until regional emergency supplies are acquired, but this cannot be achieved rapidly due to damage in MWD's regional transmission system. To counteract the loss of local storage a parallel effort is put forth into restoring flow into the City from regional storage locations at Castaic Lake (SWP) and Bouquet Canyon Reservoir (LADWP). Due to extensive LAA pipeline damage south of Bouquet Canyon Reservoir [2], attention will initially focus on restoring flows from Castaic Lake, which conceptually may be achieved within seven days. Thus, the total system serviceability will continue to decline for at least one week, but in local areas serviceability may increase from specific repair schemes. Groundwater supplies will help reduce the rate of serviceability declination. Once regional supplies are acquired total system serviceability will begin to increase, with an initial rapid service restoration rate. The restoration rate will quickly taper off due to the rationing to all the agencies needing the regional supplies for a long duration. The restoration of service to all customers may take months to achieve, primarily due to the combination of many trunk line repairs and limited emergency supplies. Tap water purification notices will remain in effect in areas throughout the system for a longer period of time until all pipes are disinfected. System repairs will continue for many more months; services are restored prior to completing all repairs by utilizing system isolation and redundancies. Restoration of normal pre-earthquake water demands in Los Angeles may take at least 15 months, and not accomplished until flows in the California Aqueduct East Branch, CRA, and regional transmission lines are restored. The lengthy demand restoration period is dependent upon the need for dispersing limited regional supplies to many agencies; thus complete restoration of the LADWP system is dependent upon the capabilities of other agencies. However, demand restoration will have significant stepped increases with increased groundwater production, when the California Aqueduct West Branch is restored after four months, and when the LAA is repaired south of Bouquet Canyon Reservoir. Even though the LADWP has a large entitlement, local groundwater pumping is not expected to meet demand, even after restoration of the West Branch, because the practical withdraw is limited due to: potential droughts that can reduce available volume prior to the earthquake, contamination, overdraft concerns, banked storage disputes, pump facility deterioration, inability to run all pumps continuously, and maintenance. During historic high drought periods, up to 30% of total demand has been met with groundwater. Using this as a reasonable amount to anticipate following the ShakeOut Scenario event, following service restoration completion, water rationing may range from 25% to 70%, depending on supplies provided by the West Branch and other MWD storage. Even with possibly the world's largest volume of potable and non-potable emergency surface storage water of any distribution agency, which is readily accessible within the distribution system, Los Angeles must undertake significant rationing efforts for long durations in the course of this disaster.

Emergency water distribution centers are strategically located throughout the City to provide customers with drinking water. The centers are slow to mobilize initially due to difficulties in communication, transportation, and limited remaining locations to acquire potable water. Distribution centers remain until the removal of all tap water purification notices. Some are maintained for many months, but the total number of distribution centers is reduced as water quality is restored.

FIRE FOLLOWING EARTHQUAKE

Scawthorn [7] outlines the ShakeOut fire following earthquake Scenario where 1,600 ignitions require fire engine response, 1,200 resulting in large fires under breezy and low humidity conditions. Hundreds of simultaneous ignitions will occur in Los Angeles. Portions of the central Los Angeles basin, where high population and building densities are subjected to severe intensity shaking, combines all the requisite factors for major conflagrations following a great San Andreas Fault earthquake. Scawthorn [7] identifies of special concern are "*portions of … the central Los Angeles basin, where the dozens to hundreds of large fires are likely to merge into dozens of conflagrations destroying tens of city blocks, and several of these merge into one or several super conflagrations destroying hundreds of city blocks*."

Many factors contribute to the spread of fire following an earthquake. The primary factor relating to water supply is fire suppression. Water is a key component in fighting fires, and in some cases, an insufficient water supply can be the differentiation between a few small fires and a major conflagration. The local water supply is most relevant to fire following earthquake. The lack of water supply adds to the number of large fires. Figure 5a identifies locations of potential ignitions as generally described by [7], overlaying the Romero [5] estimated water system flow state. The large flames in Figure 5a locate the area where [7] predicts super conflagrations. Figure 5b shows that trunk lines are unable to provide adequate water supply to this area within 24 hrs after the earthquake, a potential fire-fighting problem. Romero et. al. [5] found the Central City water service area to have a 31% serviceability, without firefighting demand. Inclusion of the large fire demand needed for a super conflagration one or more days after the earthquake could potentially degrade service to nearly zero. Although there are alternate means of fighting fires, the loss of water supply in central Los Angeles and other areas inhibits fire fighting capability and contributes to the overall mortality, injury, and economic losses in this disaster.

REGIONAL ECONOMICS

Jones et. al. [1] estimates $213.3 billion in total losses from the ShakeOut Scenario. Fire causes over $87 billion in total losses, over 40% of all economic losses. Over half of the estimated business interruption losses, approximately ¼ of all economic losses, come from water supply and distribution related problems; assuming the LAA, California Aqueduct, and CRA are fully restored to service within 6 months. The combination of fire and water account for over $140 billion in economic loss, approximately 2/3 of the total loss as estimated in [1]. However, as shown in Figure 3, Davis [2] indicates the three major water supply aqueducts may not return to service for longer than 6 months. The lack of regional storage and supplies will have a significant impact on the regional economy; the $53 billion in business interruption losses due to reduced water supply [1] may have been greatly underestimated for this event. Considering the relation between fire damage and water system damage, the earthquake effects to water supply and distribution systems has possibly the greatest impact of all aspects considered in the ShakeOut Scenario.

The Los Angeles water system is the largest, but only one of hundreds of water distribution agencies affected by the ShakeOut Scenario. The LADWP serves approximately 20% of the effected population. Therefore, water system restoration to the City of Los Angeles is important to the entire

Southern California economy. However, the regional economy does not function only through Los Angeles, it is intertwined with all the Cities impacted by the ShakeOut Scenario. Therefore, the regional economy is affected primarily through the accumulated impacts to all water suppliers and distributors in Southern California. As a result, it is critical for all water agencies to prepare for a great San Andreas Fault earthquake.

CONCLUSIONS

A review of potential damage to three major aqueducts (Los Angeles, California, and Colorado River) and the LADWP water transmission and distribution network was performed for a M7.8 southern San Andreas Fault earthquake scenario. The results indicate there may be inadequate storage to supply the local population during the length of time it takes to repair the aqueducts. Additionally, the scenario earthquake will damage LADWP pipes at nearly 2,700 locations, resulting in loss of pressure and reserve storage. The post-earthquake system serviceability declines rapidly, reaching 34% in 24 hrs. System restoration may take months due to the large number of repairs and insufficient regional emergency water supplies. A large conflagration is predicted for this scenario in central Los Angeles. Water pressures are unlikely to meet the required fire demand. Jones et. al. [1] showed that water supply impacts account for 25% of the $213 billion regional economic losses, however, this and other studies indicate the business interruption losses due to water system may be underestimated.

ACKNOWLEDGMENTS

Discussions with Dr. Ken Hudnut of the United States Geological Survey, Scott Lindvall of William Lettis and Associates, and Dr. Hadi Jonny of LADWP, are deeply appreciated. Support of the LADWP is gratefully acknowledged.

REFERENCES

[1] Jones, L.M., R. Bernknopf, D. Cox, J. Goltz, K. Hudnut, D. Mileti, S. Perry, D. Ponti, K. Porter, M. Reichle, H. Seligson, K. Shoaf, J. Treiman, and A. Wein (2008). "The ShakeOut Scenario," U.S. Geological Survey Open-File Report 2008-1150 and California Geological Survey Preliminary Report 25.

[2] Davis, C.A. (2009). "Los Angeles Water Supply Impacts from a M7.8 San Andreas Fault Earthquake Scenario," International Water Association 8th Symposium on Water Supply Technology, Kobe, Japan, paper 20039.

[3] American Water Works Association Research Foundation, AWWRF (2007). *Proc. of 5th US- Japan Workshop on Seismic Measures for Water Supply,* AWWARF/JWWA, Oakland, CA, Aug. 15-17.

[4] Metropolitan Water District of Southern California, MWD (2005). "The Regional Urban Water Management Plan," http://www.mwdh2o.com.

[5] Romero, N., T.D. O'Rourke, L.K. Nozick, C.A. Davis (2009a). "Los Angeles Water Supply Response to 7.8 M_w Earthquake," *Proc. 7th U.S. Conf. on Lifeline Earthquake Engr.,* ASCE, Oakland, June 28-July 1, paper 220.

[6] Romero, N., T.D. O'Rourke, L.K. Nozick, C.A. Davis (2009b). "Seismic hazards and Water Supply Performance," *Journal of Earthquake Engineering,* submitted.

[7] Scawthorn, C.R. (2008) "Fire Following Earthquake," ShakeOut Scenario Supplement Study prepared for United States Geologic Survey and California Geologic Survey, http://www.colorado.edu/hazards/shakeout/fire.pdf

[8] O'Rourke, T.D., J.M. Jezerski, N.A. Olson, A.L. Bonneau, M.C. Palmer, H.E. Stewart, M.J. O'Rourke (2008). "Geotechnics of Pipeline System Response to Earthquakes", *Keynote Paper, Proceedings, Geotechnical Earthquake Engineering and Soil Dynamics IV (GEESD)* Sacramento, CA, May.

[9] Environmental Protection Agency, EPA (2007). "EPANET". http://www.epa.gov/nrmrl/wswrd/dw/epanet.html

Simulation of the Restoration Process Based on Estimation of Seismic Damage to Distribution Pipes

Yuki Sagara, Katsuhiko Eguchi, Ryo Yamashita

ABSTRACT

Water supply is one of the most important lifelines for citizens, so it is essential to build a system capable of rapid restoration to minimize disruption to citizens' lives and social activities when a large-scale earthquake occurs.

Osaka Municipal Waterworks Bureau reviewed its estimation of seismic damage to distribution pipes in 2006, adding new aspects such as some outcomes of recent research in earthquake engineering. Based on this estimation, it is important to recognize in advance what actions and how many workers would be needed to restore water supply after an earthquake and how much support in terms of emergency water supply and repair activities would be needed from other cities etc.

This paper describes the method utilized for the simulation of the area and the scale of effect that would result from suspension of water supply throughout Osaka city. Moreover, as a case study, identifying the area affected by water suspension and seismic damage to pipes based on this method, we simulated the restoration process of both emergency restoration work for pipes and emergency water supply during the recovery time objective after the earthquake, and then analyzed the resources that would be required, such as the number of restoration activity personnel and the amount of emergency materials.

Yuki Sagara, Staff Officer (Earthquake Measures), Engineering Div., Osaka Municipal Waterworks Bureau, 1-14-16 Nanko-kita, Suminoe-ku, Osaka JAPAN
Katsuhiko Eguchi, Manager for Crisis Management, Engineering Div., Osaka Municipal Waterworks Bureau, 1-14-16 Nanko-kita, Suminoe-ku, Osaka JAPAN
Ryo Yamashita, Staff, Planning Dept., Engineering Div., Osaka Municipal Waterworks Bureau, 1-14-16 Nanko-kita, Suminoe-ku, Osaka JAPAN

1. INTRODUCTION

When large-scale damage is caused by earthquake or other disasters, it is important from the viewpoint of maintaining water services to promptly establish a damage control organization which includes not only personnel from the disaster area but also supporting staff from other cities. Therefore, it is necessary to predict the damage and recognize the practical recovery time objective and the scale of support to be requested from other cities in order to make swift decisions on restoration policy in the initial stage after earthquake.

Osaka Municipal Waterworks Bureau has reviewed its seismic damage estimation of distribution pipes in fiscal 2005 ~ 2006 according to some outcomes of recent research in earthquake engineering. In this paper, we calculated the seismic failure rates of distribution network pipelines in our city, and predicted the areas that would be affected by water suspension by using distribution network analysis and other measures. We also projected seismic damage of pipes from the results obtained here, and conducted a simulation to predict water suspension in the city area and in the emergency water supply centers. Furthermore, as a simulation of the restoration process in the event that an epicentral earthquake occurs in our city, we estimated transitions in the population, the emergency water supply centers affected by water suspension, and the availability of emergency water supply under the precondition of a recovery time objective of one month for the pipeline restoration process. Then, accordingly, we calculated the number of support parties to request from other cities and the number of implements and amounts of materials necessary for the emergency work.

2. RESTORATION POLICY OF OSAKA MUNICIPAL WATERWORKS BUREAU

After the Kobe Earthquake in January 1995, we created the "target of emergent water supply" and "restoration target of pipeline routes leading to respective emergency water supply centers" (refer to Table 1 and 2) as a emergent water supply scheme to reserve and secure the stable water supply that is necessary to sustain citizens' lives. These are aimed to secure the necessary amount of water required in each stage after the earthquake, such as drinking water just after the disaster and daily life water increasingly required as subsequent days pass.

Table 1. Target of emergent water supply

◆Immediately after the earthquake	…	Secure drinking water with the stocks of water cans just after the earthquake.
◆Within three days after the earthquake	…	Secure minimum water required for supporting lives, such as water for drinking and medical use.
◆From the fourth day after the earthquake	…	Sequentially and increasingly secure water to use in daily life and urban activities.
◆Up to one month after the earthquake	…	Secure normal amount of water.

Table 2. Restoration target of pipeline routes leading to respective emergency water supply centers

◆Pipelines leading to wide area shelters	…	Restore within 3 days after the earthquake
◆Pipelines leading to accommodation shelters (elementary, junior high, and high schools, etc) and important facilities (hospitals, daycare centers, etc)	…	Restore within 10 days after the earthquake
◆Pipelines leading to nearby city parks	…	Restore within 15 days after the earthquake

As shown above, we have already designated accommodation shelters for emergency water supply centers, as well as medical facilities where water delivery by water supply truck is required (hereinafter collectively referred to as "centers"). Restoration of pipeline routes leading to the respective centers should be prioritized. The period of time taken to restore normal state of water supply in the whole city area is targeted as one month at maximum.

3. OUTLINE OF THE RESTORATION PROCESS SIMULATION

Figure 1 shows an outline of the restoration process simulation which we developed in this paper. At first, we estimated seismic failure rate of distribution pipelines in the city area using a seismic damage estimation formula that we had revised according to some outcomes of recent research in earthquake engineering.

Secondly, using the estimated seismic failure rate as a parameter, we combined the result of a distribution network analysis, which took leakage from main pipelines (400 mm and more in diameter, hereinafter referred to as "mainline") into account using the Monte Carlo method, with a statistical estimation model of water suspension in the majority of other distribution pipes (less than 400mm in diameter, hereinafter referred to as "small pipes"), and estimated water suspension situation before starting the restoration work in the city area.

Finally, as a case study, we simulated the restoration process of both emergency restoration work on pipes and emergency water supply based on the estimated results of the pipeline damage or water suspension situation, taking in a simulation of water suspension area where the suspension was reduced day by day as restoration work progressed according to the restoration policy. We obtained a time series prediction of restoration process by repeating this procedure on each day from occurrence of the disaster to the end of restoration on the condition of our time objective (one month) for recovery.

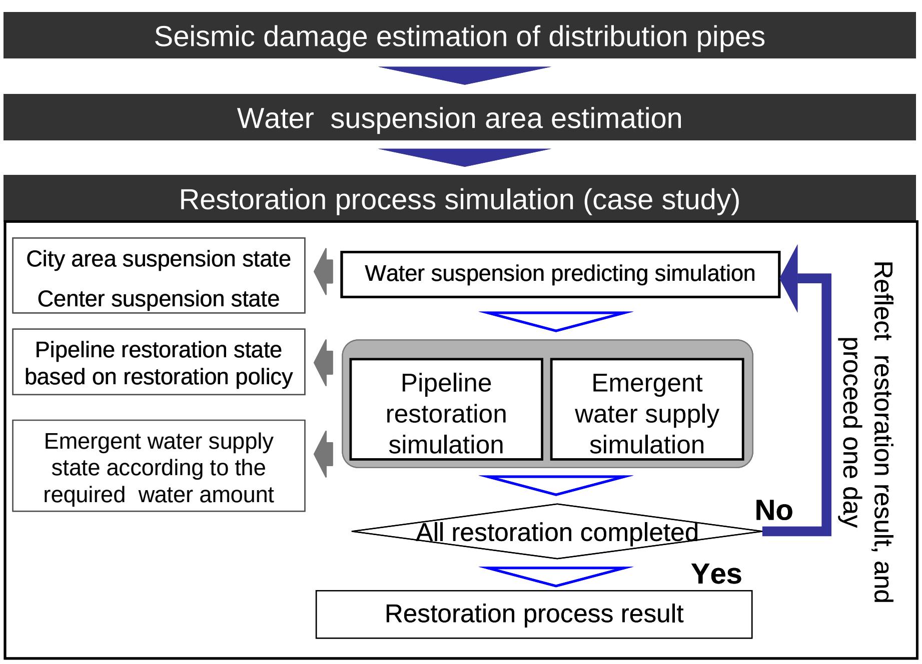


Figure 1. Outline of the restoration process simulation

4. ESTIMATION OF SEISMIC DAMAGE TO DISTRIBUTION PIPES IN THE CITY AREA

In this study, we carried out seismic damage estimation of distribution pipes in the city area for the Uemachi fault earthquake, which is assumed to cause the largest damage of the earthquake scenarios considered in Osaka City Local Area Disaster Protection Plan. The city area is divided into a grid composed of 3,691 units of 250 m x 250m cells. In this prediction, we added some outcomes of recent research in earthquake engineering as a new point of view, applying the simulated seismic motion of the 1995 Kobe Earthquake to seismic damage estimation, reflecting seismic damage characteristics of pipelines by strong seismic motion, and evaluating degrees of liquefaction to estimate liquefied ground damage. We used the seismic damage estimation formula to estimate seismic failure rate for each pipe type and diameter by means of peak ground velocity (PGV) and potential of liquefaction (PL value)[1].

The seismic damage estimation formula for distribution pipes used here is as follows:

$$DI = C_d \times \{DI0 \times (1 - A_L) + DI0_L\} + DI0_L \times A_L \tag{1}$$

Wherein,

DI: Seismic failure rate of cell (failure/km)
DI0: Seismic failure rate in non-liquefaction ground (failure/km)
$DI0_L$: Average seismic failure rate in liquefaction ground (failure/km)
A_L: Liquefaction area factor of cell
C_d: Correction coefficient based on inner diameter

Table 3. Seismic failure rate in non-liquefaction ground (DI0)

Pipe type	Seismic failure rate estimation formula	
DIP-S	$DI0=0$	
DIP	$DI0=0.0056\,(PGV-15)$	(PGV<150)
	$DI0=0.7560$	(PGV>=150)
CIP	$DI0=0.0232\,(PGV-15)$	(PGV<120)
	$DI0=2.4360$	(PGV>=120)
VP	$DI0=0.0177\,(PGV-15)$	(PGV<120)
	$DI0=1.8585$	(PGV>=120)
SP	$DI0=0.0043\,(PGV-15)$	(PGV<150)
	$DI0=0.5805$	(PGV>=150)

Table 4. Seismic failure rate in liquefaction ground ($DI0_L$)

Pipe type	Seismic failure rate
DIP-S	$DI0_L=0$
DIP	$DI0_L=2.56$
CIP	$DI0_L=4.00$
VP	$DI0_L=1.83$
SP	$DI0_L=0.97$

Table 5. Correction coefficients based on inner diameter (C_d)

Diameter (mm)	DIP,SP	CIP	VP
～75	2.1	1.7	1.1
100～150	1.0	1.2	0.8
200～250	1.0	1.1	-
300～450	1.0	0.6	-
500～	0.2	0.2	-

As a result of applying the above formula to Osaka city area, the number of damaged pipelines came to about 5,900 points. The seismic failure rate of distribution pipes is shown in Figure 2.

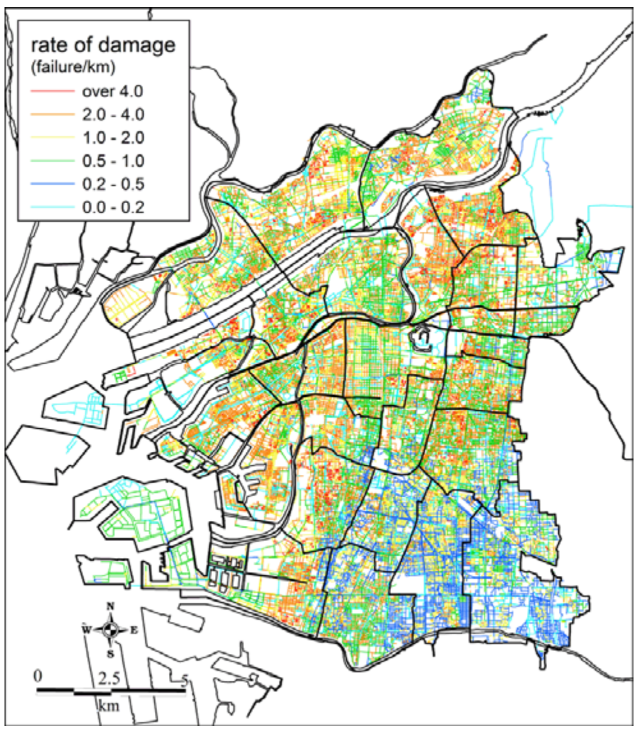


Figure 2. Seismic failure rate of distribution pipes

5. PREDICTION OF WATER SUSPENSION AREA IN THE CITY

We predicted the water supply suspension area in the city immediately after the earthquake, combining the distribution network analysis result considering leakage in mainline and the statistical estimation model of water supply suspension in small pipes.

At first, based on the predicted result of seismic failure, we estimated the rate of water suspension in the mainline using the distribution network analysis, which took leakage into account using the Monte Carlo method. The number of damaged pipelines calculated in the previous section is the total sum of values which are derived from pipeline extension multiplied by the seismic failure rate estimated by grid, pipe type, and diameter, respectively, so this figure does not indicate specific locations of damage points. Thus we randomly applied the distribution of pipe damage points on the pipeline network model by means of random numbers as a condition, weighing the distribution of the seismic failure rate, and obtained the water pressure on nodes of mainline network model by means of the distribution network analysis. When water pressure became lower than 0.04 MPa, we determined it as water suspension. We ran the simulation 5,000 times, and the rate of occurring water suspension in the trials at nodes of mainline network models in each grid was determined as the water suspension rate of mainline in a grid according to the following formula.

$$D^{[m]} = D_i^{[m]} = N_i / M \tag{2}$$

$D^{[m]}$: Water suspension rate of mainline by grid

$D_i^{[m]}$: Water suspension rate at a node (i) in or nearest to a grid

N_i : The number of times that the node (i) became water suspension

M : Number of simulations conducted (5,000)

As pipes for transmission and distribution are laid all over the city and total more than 5,000 km of overall extension, it is not practical to apply the distribution network analysis or modeling to small pipes. Thus, we estimated the water suspension rate of small pipes at one day after the earthquake from the following formula which shows correlation[2)] between the seismic failure rate and the water suspension rate based on recent examples of earthquakes including the Kobe Earthquake.

$$y = 1/(1 + 0.307x^{-1.17}) \tag{3}$$

Wherein,

y : Water suspension rate from one day after earthquake

x : Average seismic failure rate per grid

Considering the influence of the relation between upstream (mainline) and downstream (small pipes) on water suspension, we assumed cases of suspension in mainline and cases of suspension in small pipes though mainline was not damaged. Therefore, we combined water suspension rate in mainline and that in small pipes according to the following formula to derive the water suspension rate in a grid.

$$D = D^{[m]} + (1 - D^{[m]})D^{[b]} \tag{4}$$

Wherein,

D : Water suspension rate

$D^{[m]}$: Mainline water suspension rate

$D^{[b]}$: Small pipes water suspension rate

As a result of applying the above formula to Osaka city area, the water suspension rate in the city was 77%. The predicted water suspension area is shown in Figure 3.

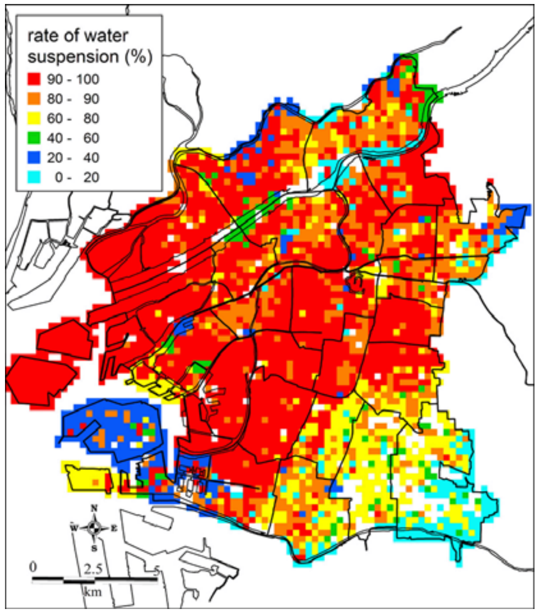


Figure 3. Prediction result of water suspension area

6. SIMULATION OF THE RESTORATION PROCESS JUST AFTER THE EARTHQUAKE

For this study, we conducted simulation of the restoration process in the water suspension area estimated in the former section (Figure 3) in the case of the assumed epicentral earthquake, and presented the most reproducible case among the results calculated 5,000 times by the Monte Carlo method as a case study. Concretely, we estimated the transition of the population in water suspension, the emergency water supply centers, and of the amount of temporal water supply during the recovery time after the earthquake, based on the most reproducible result of water suspension area prediction. Then we evaluated the number of support parties that it would be necessary to request from other cities and the number of implements and amount of materials needed for emergency work, conducting time series simulation of water suspension situation in the city and the emergency water supply centers during the recovery time objective (one month).

6-1 Simulation of water suspension predicting

Separating the case of mainline and small pipes, we simulated the water suspension situation in the city for the period from the earthquake event to the end of restoration work.

As for the mainline, we estimated the distribution of water pressure by using the distribution network analysis considering leakage as mentioned above.

As for the small pipes water suspension rate from the second day after the earthquake, the damaged points are cut off from the distribution network by the sluice valves within a certain time from the earthquake. So the influence of water suspension in small pipes is limited within the damaged pipeline section. Thus, we estimated the small pipes water suspension rate from the rate of damaged points in the pipe section separated by the sluice valves in a grid by using the following formula.

$$D^{[b]}(i) = N_d(i) / B(i) \tag{5}$$

$D^{[b]}(i)$: Small pipes water suspension rate in a grid (i)

$N_d(i)$: Number of damaged points in small pipes in a grid(i)

$B(i)$: Number of sections between sluice valves in a grid(i)

Then we judged the water suspension situation for each grid by integrating the relation between upstream, that is mainline, and downstream, that is small pipes (refer to the formula (4)), into these results. The population in water suspension in a grid was obtained by multiplying the water suspension rate in a grid by the population.

As for centers, we judged the water suspension situation for the respective centers. Searching the shortest route by Dijkstra's Algorithm considering the seismic damage rate of distribution pipes and hydraulic resistance, we set up five routes in each center that are to be restored preferentially. If damage was found in a small pipe on the preferential route, we judged it as water suspension, and if not, we determined the degrees of water suspension according to the water pressure state in the connected mainline.

6-2 Simulation of emergency restoration work of pipes

In the simulation of emergency restoration work of pipes, we basically gave priority to the restoration of mainline and the pipeline leading to the centers in water suspension condition according to the restoration policy of Osaka city. A flow chart for determining restoration points is shown in Figure 4. Priority in restoring centers was first given to the wide area shelters, then emergency hospitals, medical facilities for treating dialysis, and accommodation shelters. As for mainline, we gave priority to restoring the damaged points that could recover more water supply when restoration works were finished. We defined restoration speed for mainline and small pipes respectively, based on our actual achievements (Table 6). We determined to offer 28 parties a day for the restoration work from our city and, trying various numbers to input for the support parties from other cities, we calculated the optimum number of restoration support parties necessary to complete restoration works within one month of our ultimate target. In this case, we assumed keeping the number of the restoration support parties after the arrival of requested support parties, within five days from the earthquake following to the beginning of restoration work on the third day, until the restoration work was completed, when considering the reinforced support system after the Kobe Earthquake and actual performance in recent earthquakes (Figure 5).

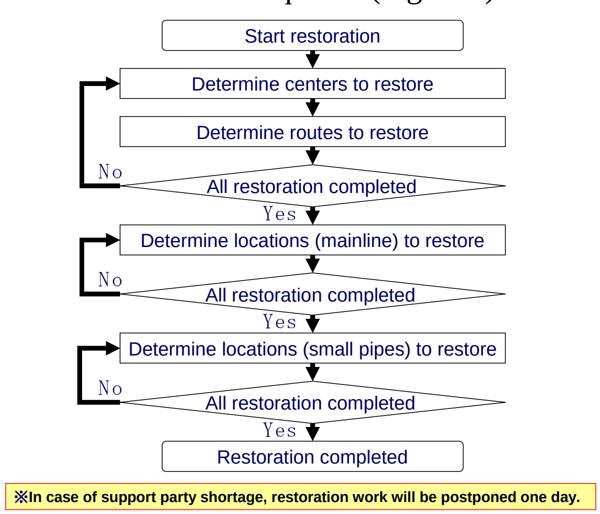


Figure 4. Flow chart to determine restoration points

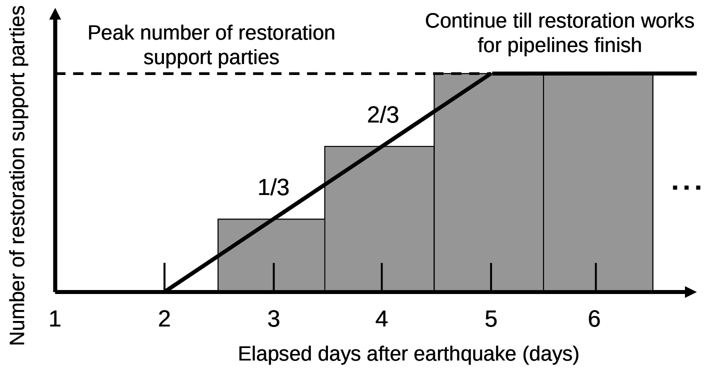


Figure 5.Transition of the restoration support parties

Table 6. Restoration speed per one party

Mainline restoration	0.25 point / day
Small pipe restoration	2 points / day

Figure 6 shows the result of the pipeline restoration simulation, and Figure 7 shows the transition of the rate of damaged points in mainline and small pipes. Based on this result, we estimated the number of support parties necessary to restore normal water service within one month of our target to be 122 parties a day maximum and about 3,300 parties in total. We found that support parties from other cities played a very important role, and that the time objective for restoration work was appropriate because the resultant support scale was feasible in terms of various support agreements. We could reproduce the situation that the population with water suspension decreased as the restoration of small pipes progressed after the mainline had been restored preferentially and water pressure had been recovered. We also found that water supply was preferentially restored in the centers, because water suspension was solved in every center within nine days after the disaster.

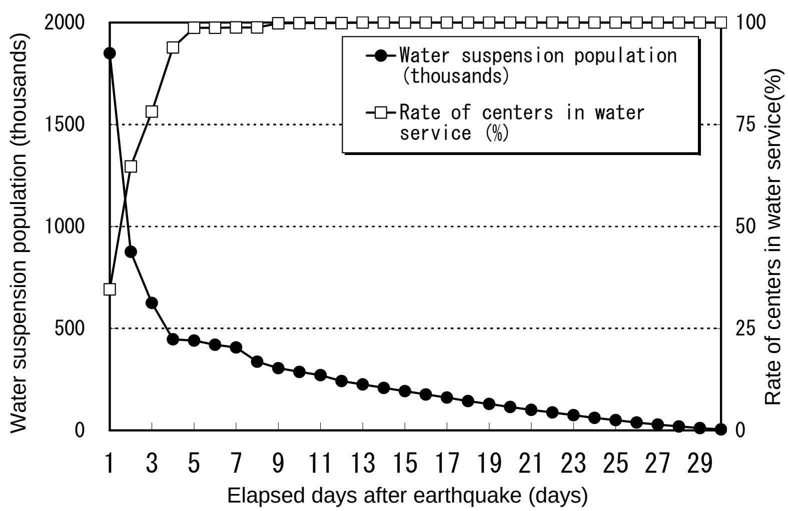


Figure 6. Result of the pipeline restoration simulation

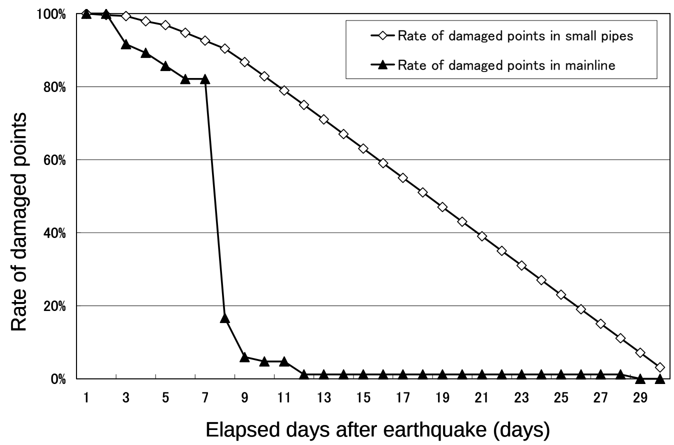


Figure7. Change in the rate of damaged points in main pipeline and small pipes

6-3 Simulation of emergency water supply

In the simulation of emergency water supply, we basically supplied a required amount of water according to the water suspension situation in the city. The flow chart of the simulation of emergency water supply is shown in Figure 8. The amount of water required for the citizens was estimated on the basis of the population with water suspension and according to the target amount of water in Table 7. The amount of water delivered to hospitals and other medical facilities was estimated on the basis of water suspension situation in the facilities and using the required unit amount of water which had been respectively predetermined for dialysis and other medical treatments. With considering the capacity of emergent water-supply equipment and the distance to convey water for the citizens, we conducted simulation to supply water with temporary tanks and delivery to the centers under water suspension and to supply water with temporary water taps or center taps to install in the centers where nearby distribution pipes are restored or not damaged. We assumed from experience in recent earthquake cases that shortage of required water would be supplied by the requested support parties which would arrive in two days after the disaster. For immediately after the earthquake, we assumed the use of seismic storage tanks and stocks of water cans before supporting parties started full scale work.

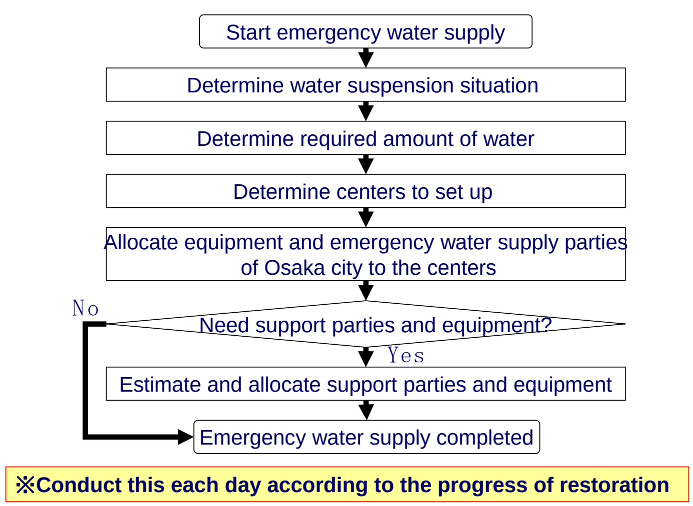


Figure 8. Flow chart of emergency water supply simulation

Table 7. Condition of emergency water supply simulation

Days elapsed after earthquake	Target water unit amount	Water conveying distance for citizens
In 3 days	3 L/person a day	Within approximately 1 km
In 10 days	20 L/person a day	Within approximately 250 m
In 21 days	100 L/person a day	Within approximately 100 m
From 22nd days	250 L/person a day	Within approximately 100 m

The time series result of emergent water-supply simulation is shown in Figure 9. From this result, we estimated required water amount and the required number of support parties for water supply according to the target amount and the status of restoration. The required amount of emergency water supply increased on the fourth, eleventh, and twenty-second days. This is because, as shown in Table 7, the target amount of water for each citizen increases as each day passes following the earthquake. Figure 10 and 11 show the result of the number of units needed of emergent water-supply equipment. As is shown in these figures, we could reproduce the situation that temporary water storage tanks and water delivery were utilized immediately after the earthquake due to water suspension, and then temporary water taps were installed and increased in numbers as the restoration work proceeded in the centers, and in turn decreased as the restoration work finished.

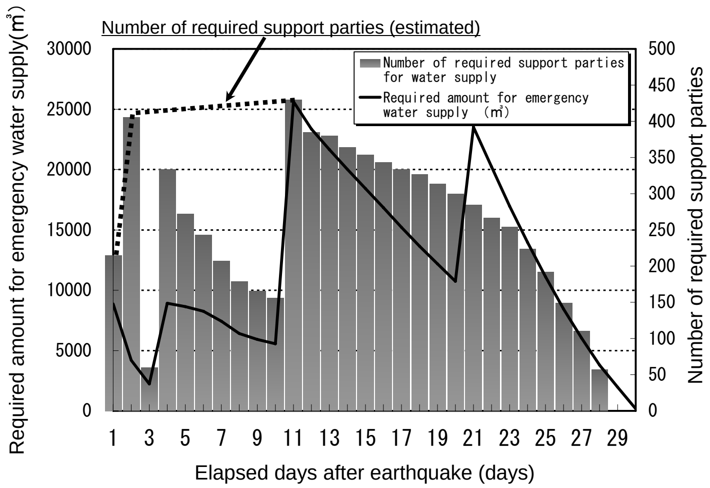


Figure 9.Result of emergency water supply simulation

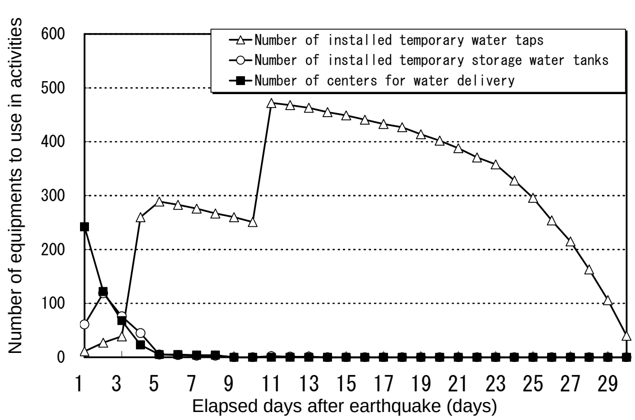


Figure10. The number of units of required emergency water supply equipment

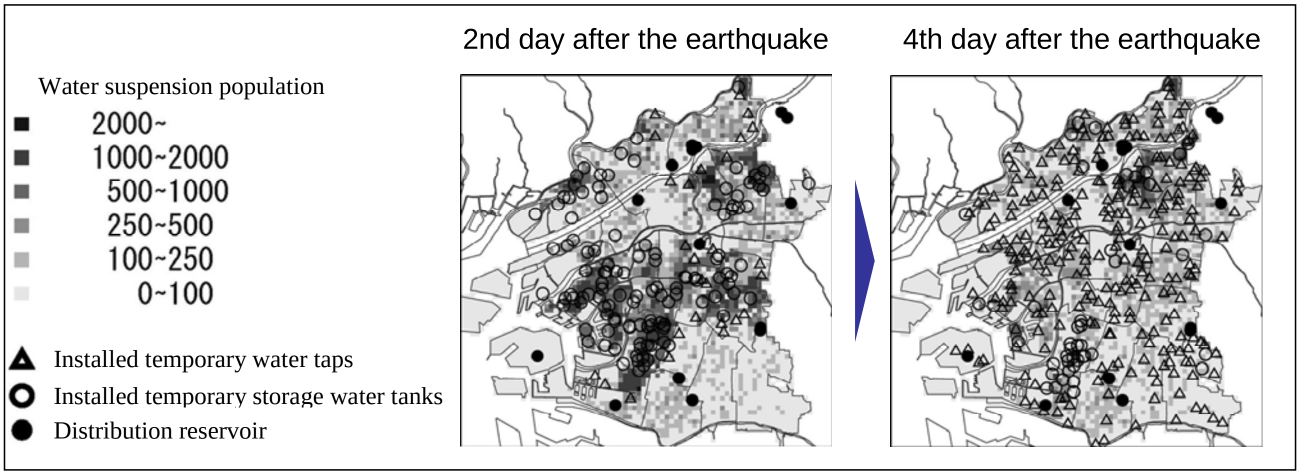


Figure 11. Distribution of installed emergency water supply equipment (2nd and 4th day after the earthquake)

7. CONCLUSION

We developed methods to predict the water suspension area and to simulate the restoration status from the occurrence of the earthquake to the end of restoration process, estimating pipeline damage distribution by means of the seismic damage estimation formula of distribution pipes. We estimated the transition of the population in water suspension, the number of the centers having water service, and the amount of emergent water supply, as well as the number of support parties to be requested and the materials and implements required. These results can be used as grounds to make swift decisions on the restoration target and the criteria for requesting support parties immediately after the disaster. They can also be used as a guideline to control the restoration progress even after the early stage of restoration work. Furthermore, this simulation can be used to evaluate the effectiveness of an earthquake prevention project by applying this simulation to earthquake countermeasure models. In the future, we are planning to examine official works in emergent situations of disaster to explore the way to utilize Osaka City Waterworks Bureau personnel effectively as well as the support parties with the aim of improving capability to continue provision of our water service.

REFERENCES

[1] H.Miyazaki, T.Nakai, Y.Komatsu, K.Yamano. 2007. "Seismic Damage Estimation of Distribution Pipes," presented at the 5th Workshop on Seismic Measures for Water Supply, August 15-17, 2007.
[2] Eiji Kawakami. 1996, The first comprehensive symposium for city epicentral earthquake disaster, pp.169-172.
[3] Jyugo Fuji. 2002, Graph network and combination theory, Kyoritsu publication, p. 206.
[4] Explanation of Project guidelines for earthquake-resistant water service (draft). 1997, Japan Water Research Center, p.33.

The Quantitative Risk Analysis of Water Pipe Bridge on Earthquake Disaster - Case Study of Sindian Water Pipe Bridge

Ban-Jwu Shih, Siao-Syun Ke, Pei-Chung Hsu and Wei-Chin Chen

ABSTRACT

Earthquake is characterized by unpredictable and destructive, compared to other natural disasters. In order to enhance the effectiveness and reliability of seismic management measures, this study combined quantitative risk analysis techniques (event tree analysis and fault tree analysis) to carry out water pipe bridge seismic safety assessment.

Based on the chief consideration that to maintenance water supply in the aftermath of the earthquake, this paper deduced the possible failure events (damage situation) of the water pipe bridge caused by the earthquake through the event tree analysis, and analyzed possible failure elements of each failure event through the fault tree analysis. Then, the important risk factors could be prioritized by the results of quantitative analysis.

In this study, the probability of each failure element mainly was analyzed by the following two ways:

1. The probability of the failure element related to structural damage was analyzed through the vulnerability of the bridge.
2. The probability of the failure element related to material damage was analyzed through the reliability of the material.

At last, this paper carry on the case study of Sindian water pipe bridge as reference.

Keywords: Water Pipe Bridge, Seismic Assessment, Quantitative Risk Analysis

INTRODUCTION

Taiwan is located at the collision area between Philippine Sea Plate and Eurasian Plate, therefore earthquake is one of the main threats for bridge safety in Taiwan, especially to water pipe bridge which is among the most important life facilities. Water can not be transported normally if the pipes break off during earthquake and the damage it brought to the society and economy is hard to estimate. Currently the safety estimation for water pipe bridges in Taiwan is mainly based on seismic resistance factor.

Ban-Jwu Shih, Associate Professor, Department of Civil Engineering, National Taipei University of Technology
Siao-Syun Ke, Assistant Researcher, Management System and Policy Division, National Science and Technology Center for Disaster Reduction, Taiwan
Pei-Chung Hsu, Secretary General, Chinese Taiwan Water Works Association
Wei-Chin Chen, Graduate Student, Department of Civil Engineering, National Taipei University of Technology

Taipei Water Department proceeded a safety inspection and evaluation on 4 water pipe bridges within its control area by D.E.R.&U visual inspection method. Beside co-operated with National Bridge Maintenance Management System to use D.E.R.&U and ABCD system as its framework, it also set up the evaluation indicators and their weight in the first place to choose the subject for preliminary evaluation [1].

Within the "Seismic Design Standards of Highway Bridge" distributed by the Ministry of Transportation and Communication (MOTC), it is stated that the seismic resistance analysis of a bridge is based on site inspection data, bridge actual dimensions, reinforcements, and the material strength, which can be combined with seismic resistance analysis on bridge structures to determine various failure modes and the corresponding earthquake surface acceleration.

In current years, several related researches estimate structural intensity and its life-span based on risk factor to increase estimation reliability such as seismic vulnerability curve analysis for bridge. Disaster triggering factors on bridge structural damage caused by earthquake can be roughly divided to ground vibration and soil layer failure, where the seismic intensity on ground surface is indicated by Peak Ground Acceleration (PGA), and the degree of failure on soil layer is indicated by Permanent Ground Displacement (PGD) which is induced by earthquake.

Risk analysis techniques are used in this research on the seismic safety evaluation of bridges combined with other methods such as Event Tree and Fault Tree to calculate the occurrence and failure probability through basic statistics factors such as failure rate data, to understand the damage risk of water pipe bridges after earthquake.

QUANTITATIVE RISK ANALYSIS OF WATER PIPE BRIDGE

The Seismic Risk Analysis Model of Bridge and the introduction of SinDian Water Pipe Bridge

Seismic safety evaluation of bridge combined with quantitative risk analysis techniques are used in this research with the case study of water pipe bridge in SinDian. Event Tree is established in the first place including occurred earthquakes, and then the events occurrence probability will be calculated through events such as "whether or not the soil liquefaction has happened", "whether or not the bridge has damaged", "whether or not the anchor has damaged", and "whether or not water pipe has damaged", then furthermore deduce the occurrence reasons of the failures occurred on the Event Tree by Fault Tree methods [2]. FaultTree+V11.20 software is used in this research to proceed the parameters quantification through various basic fault extinguishment in Fault Tree, then calculate the extinguishment rate of fault events and the occurrence probability of events in Event Tree to later analyze the related results.

The water pipe bridge in SinDian is taken as the case study of this research to proceed the seismic safety evaluation with quantitative risk analysis. This water pipe bridge is the second fresh water transportation pipe managed by the Taipei Water Department, which crossed SinDian river from AnKang district to Jhongyang Village. It is a steel arch bridge with length of 290m, which upper structure consists of 3-span steel arches, each is 70m long. Arches height is 10.6m, and there are also 11 shafts supporting the main water pipe. Lower structure consists of a 12m-high single reinforced concrete pier with pile foundation. The inner diameter of water-transporting pipe is 2400mm, with 20mm thickness. The structure of SinDian water pipe bridge is shown in Figure1.

Figure 1: The structure of SinDian Water Pipe Bridge

The Establishment of Event Tree and Fault Tree

The main method to proceed the Quantitative Risk Analysis is by establishing Event Tree and Fault Tree to find out the probable fault events and the main cause of the events, combined with parameters quantification to calculate the failure probability of each event, we can then proceed maintenance to the events with higher failure probability.

The establishment of Event Tree is started from the occurrence of earthquake to the failure of water-transporting pipe, then we can deduce the event from the occurrence to its conclusion. Because earthquake is developed bottom-up, therefore it goes from soil liquefaction through bridge failure, anchor failure and water pipe failure in the end. Event Tree is established under the consideration whether or not the function of water pipe has failed, thus we determine the mutual influence between water pipe and bridge to calculate events which will probably fail such as “whether or not the soil liquefaction has happened”, “whether or not the bridge has damaged”, “whether or not the anchor has damaged”, and “whether or not water pipe has damaged”.

The reasons of failed events will be probed after the establishment of Event Tree to set up the Fault Tree. Fault Tree will be analyzed from 2 factors: “Substantial damage” and “Damage caused by external force” to find out whether the failed events were caused by single basic fault or several basic faults collectively [3]. The considerations for constructing Fault Tree are as below:

- Since the bridge discussed here is different from usual bridge where the structure above the pier is water pipe, not usual beam and slab, with steel arches to resist the moment, therefore considerations for the failure parts of this bridge will be divided to underground foundation and upper ground structures:

a. Basic failure: foundation settlement which is mainly caused by soil liquefaction and ground dislocation; and foundation displacement which is mainly caused by lateral sliding and tilting of the ground.

b. Structural failure: upper structure and lower structure failure.

(a) Upper structure failure includes steel arches failure and bridge collapse. Steel arches failure can again be separated to over-deformation failure (displacement, tilting and deformation failure) and failure by external force (flexure, shear and flexure shear failure).

(b) Lower structure failure includes bridge abutment and pier failure.

i. Bridge abutment failure includes ground dislocation, loose backfill, and soil liquefaction.

ii. Pier failure includes over-deformation failure (displacement, tilting and deformation failure) and failure by external force (flexure, shear and flexure shear failure).

- Anchor is the connecting part, the fixed end and also the surface preserver of the bridge. It is divided to U-shaped steel ring failure, bolt failure and coating degradation.

  a. U-shaped steel ring failure: the substantial defect is mainly by corrosion, when the failure caused by external force is mainly by shear failure and over-deformation.

  b. Bolt failure: bolt defect is mainly related to material quality such as corrosion, temperature difference, ageing, inferior material etc, when the failure caused by external force is shear and pull-out failure.

  c. Coating degradation: Anchor is usually exposed under sunlight, wind and rain, thus coating degradation such as peel off, swollen, crack, color fading and rust is unavoidable.

- In the aspect of water pipe bridge losing its main function due to pipe failure, it can be differed to joint failure, pipe body damage and fittings failure.

  a. Joint failure: joint defects are mainly caused by inferior material, when the failure caused by external force consists of tension failure, bearing failure, water pressure failure and flexure failure.

  b. Pipe body failure: Pipe body defects are mainly caused by material defects such as splits, temperature difference, hardening and inferior material quality. The failure caused by external force consists of cracks, twisting/deformation, cut-off and water pressure failure.

  c. Fittings failure: includes the failure of pipe-fixing facility, cracks on manhole or loosened anchor bolts.

Parameters Quantification Method

Parameters quantification method used in this research to calculate the occurrence probability of basic failure events in the Fault Tree is mainly differed to 2 parts: failure probability of bridge structures analyzed from vulnerability aspect, and the failure probability of the material analyzed from reliability aspect.

Failure probability of bridge structures analyzed from vulnerability aspect

Within "Seismic Capability Evaluation and Reinforcement Project Feasibility for Highway Bridges" by CECI Engineering Consultants, Inc., there are 148 representative bridges chosen from bridges along the provincial highways in Taiwan to be evaluated in the aspect of their seismic capability, which are put into a complete report of seismic vulnerability analysis [4]. The outcome of this report refers to bridge classification in TELES, which provides the average value and correction method for the Vulnerability Curve parameters (median and standard deviation) for various type of bridge, including Peak Ground Acceleration (PGA)-Seismic Vulnerability Curve, and Displacement-Seismic Vulnerability Curve, which enable us to establish the Seismic Vulnerability Curve for bridges conveniently. By this method, we establish PGA-Seismic Vulnerability Curve diagram and Displacement-Seismic Vulnerability Curve diagram, then we calculate the failure probability when the PGA is 1g and Permanent Ground Displacement (PGD) is 0.2m, with related considerations as below:

- The PGA of Chi-Chi earthquake is 986cm/s^2, about 1g, therefore it will be used as the occurring earthquake intensity. The failure probability ranges from light damage with 0.907 probability, medium damage with 0.699 probability, severe damage with 0.313 probability, to total damage with 0.073 probability.

- Based on the damage rate of underground pipes after earthquake analyzed in "A Study on Earthquake Risk to Underground Fresh Water Pipes in Taipei" [5], we choose 0.2m as probable occurring circumstance, which failure probability ranges from light damage with 0.971 probability, medium damage with 0.790 probability, severe damage with 0.391 probability, to total damage with 0.087 probability.

Failure probability of steel material analyzed from the aspect of material reliability

We consider the material reliability on 4 different usage phases of steel material in the water pipe bridge, which can be differed to 4 phases of the usage time normalization (usage time/mean life), which are: 0.25, 0.50, 0.75 and 1.00. Material reliability will degrade over time, as shown in Figure 2, where material reliability on these 4 phases is 0.915, 0.728, 0.542 and 0.4000 accordingly, or in another word, the extinguishment rate is 0.085, 0.272, 0.458 and 0.600.[6]

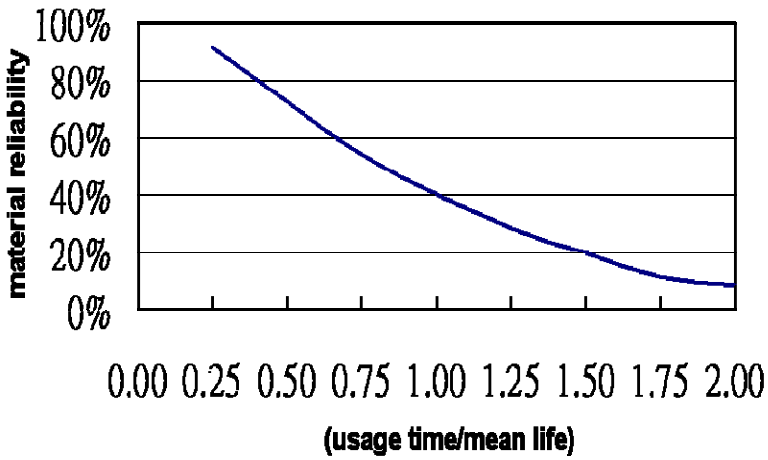


Figure 2: The chart of steel material reliability[6]

QUANTITATIVE RISK ANALYSIS RESULT FOR WATER PIPE BRIDGE

The Fault Tree of bridge, anchor, and water pipe damage, along with Event Tree calculated by FaultTree+V11.20 are shown in Figure 3 and Figure 4 below.

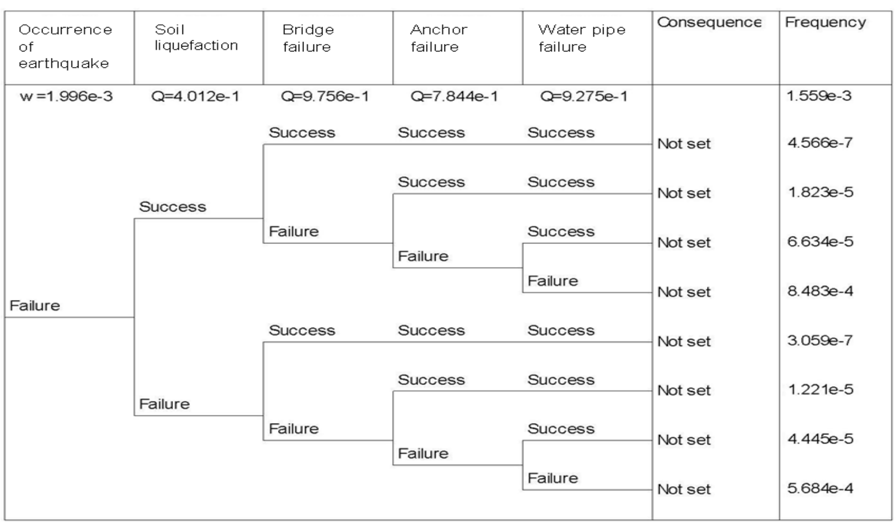


Figure 3: The event tree of SinDian Water Pipe Bridge (usage time normalization = 0.25)

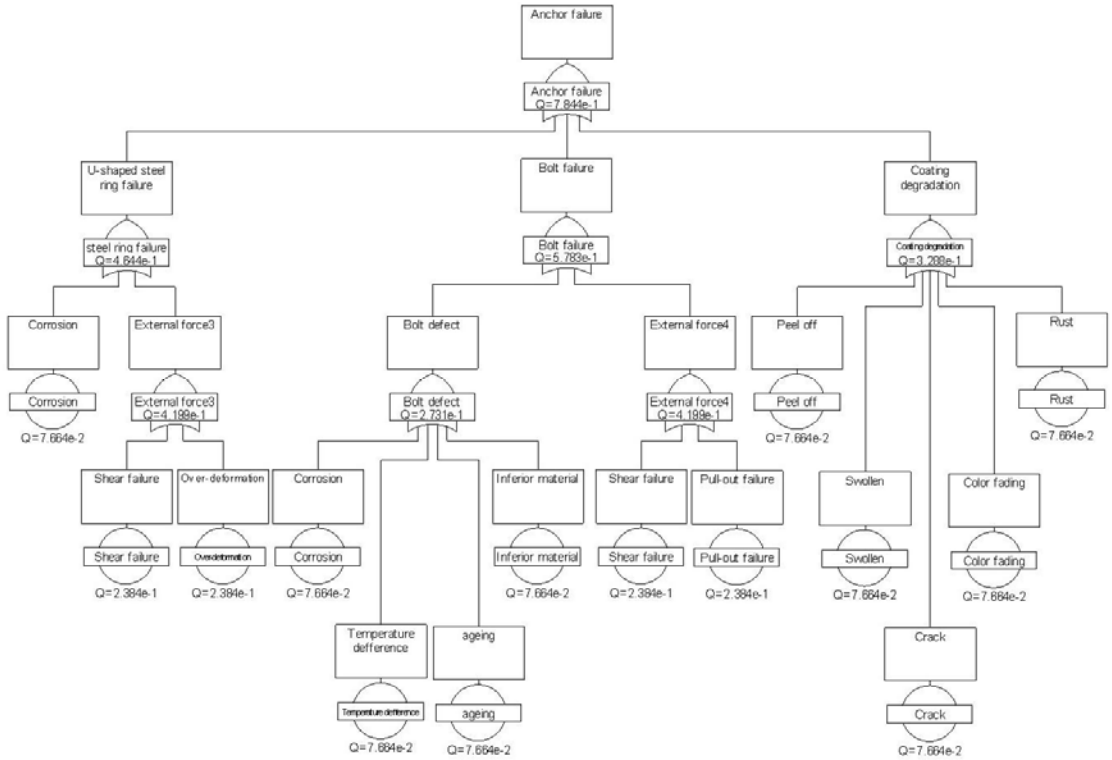


Figure 4: The fault tree of anchor failure (usage time normalization = 0.25)

The main outcomes of the quantification of failed events probability for water pipe bridge in this research are as below:

- In Event Tree, the usage period normalization for steel material reliability ranges from 0.25 to 1.00, and the highest failure probability value of water-transporting function is 8.483e-4, 1.088e-3, 1.143e-3 and 1.156e-3. The trend of extinguishment rate of water-transporting function is shown as Figure 5.

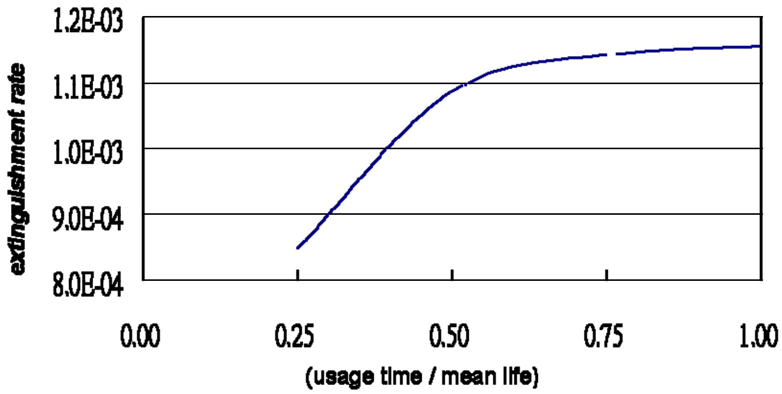


Figure 5: Trend chart of extinguishment rate of water-transporting function

- Higher failure probability indicates that the reliability and substantial strength of steel material is degrading over time, which increases the damage rate of water pipe. The increasing curve of failure probability over time tends to flatten when the usage period normalization is 0.5 or above.
- From Event Tree analysis result we found out that the controlling factors for usage period normalization which ranges from 0.25 to 1.00 are bridge and anchor failure, and from Fault Tree analysis for anchor failure it shows that when the usage period normalization is 0.5 or above, the controlling factor for bolt failure changes from failure caused by external force to bolt defects. These results can assist the management unit in planning inspection and maintenance priorities for different period.

CONCLUSION

In this research we establish Event Tree and Fault Tree of water pipe bridge after earthquake and propose parameters quantification method for various basic failure events with SinDian water pipe bridge as the case study. The evaluation and analysis results show that extinguishment probability of related failure events estimated through Vulnerability Curve for bridge and material reliability analysis could provide simple quantification data which can be used conveniently for preliminary evaluation.

The difference of Quantitative Risk Analysis method in comparison with the actual estimation methods is, in this method overall risk will be considered coordinatively. This method could consider the condition of various components under the sequence of circumstances within a disaster and establish distinctive Event Tree and Fault Tree according to each protection target. In the same time, the steel material reliability variation over time is also considered in the evaluation process to analyze risk management priorities of the protection target under different life cycle.

REFERENCES

[1] Yang-Lung Wu & Chuan-Chiang Fan & Chao-Ming Chen (2006), “Pipe Bridge Safety Inspection and Assessment,” Water Conference, Vol.25, No.4. (*In Chinese*)

[2] Ching- Hsien Huang (1999), *Hazard Analysis and Risk Assessment*, San Min Book Co, Ltd, Taipei city.(*In Chinese*)

[3] PCC, Executive Yuan(1999), *Bridge Structures after The Earthquake and the Reinforcement of Rapid Diagnostic Manual- Bridge Structures after the Earthquake Rapid Diagnostic manual of the Establishment and Rapid Post-Earthquake Reinforcement Means of a Bridge*, Taipei city. (*In Chinese*)

[4] CECI Engineering consultants, Int., Taiwan(2008), *Seismic Capacity Evaluation of Highway Bridges and Reinforced Engineering Feasibility Study*, Directorate General of Highways, M.O.T.C., Taipei city. (*In Chinese*)

[5] Ban-Jwu Shih & Chin-Hsun Yeh & Gee-Yu Liu(2006), *Taipei Water Department for Further Investigation at the End of the Report - the Underground Water Supply Pipeline Taipei earthquake Criticality Study*, NCREE, Taipei city. (*In Chinese*)

[6] Fang-Hsun Hsu(2001), *The Study of Dynamic Reliability Models and Its Application,* doctoral dissertation, NCU, Taoyuan county. (*In Chinese*)

A Reasonable Approach to the Seismic Assessment of Water Supply Facilities

Hayato Yokono and Akinori Sakata

ABSTRACT

The majority of Kobe City's water supply facilities were developed from the mid-1960s to the mid-1970s, and at present, efficient renewal and seismic reinforcement of these facilities on a limited budget poses a major challenge. Moreover, because these facilities will face renewal at the same time in the near future, the strategic management of assets by leveling investments and making investment plan decisions according to revenues, etc. is very important.

As such, Kobe City has undertaken diagnostics and comprehensive assessment of its distribution pipe networks, resulting in the "Pipe Network Diagnosis and Evaluation System (P-DES)," which has already been put into operation. For distribution reservoirs and other key facilities, on the other hand, Kobe City has constructed the "Water Supply System Reliability Assessment Program," which evaluates the reliability of the entire waterworks system and sets the renewal priority level of each facility. At the same time, the city is conducting diagnosis and evaluation of the earthquake resistance and degradation level of one facility after another in order to determine their current condition.

This paper reports on the reasonable approach that has been adopted for proper evaluation of unique conditions, such as ground properties and structural characteristics, etc., in the series of programs for seismic assessment of existing distribution reservoirs.

Hayato Yokono, Maneger, Planning and Design Division, Kobe City Waterworks Bureau, 6-5-1, Kano-cho, Chuo-ku, Kobe, 650-8570, Japan

Akinori Sakata, Design Cief, Planning and Design Division, Kobe City Waterworks Bureau, 6-5-1, Kano-cho, Chuo-ku, Kobe, 650-8570, Japan

1. INTRODUCTION

Since the beginning of its water services in 1900, Kobe City's water supply facilities have constantly developed and expanded along with the rise of urbanization. Geographically, the city is bisected horizontally by Mt. Rokko, with the south side consisting of a belt-like urban area with a large difference in elevation, and the north side being made up of very hilly terrain. Taking advantage of its undulating topography, the water system of Kobe City uses a gravity flow system, in which distribution reservoirs are constructed at different elevations. Because of this, Kobe City's water supply business is characterized by the large number of its facilities. Since most of these facilities were built between 1965 and 1980, their efficient renewal and seismic reinforcement within a limited budget pose a major challenge for the city. In addition, it is important to strategically manage assets by leveling investments and making investment plans according to revenues in order to meet future capital requirements. Therefore, Kobe City has undertaken diagnostics and comprehensive assessment of its distribution pipe networks, resulting in the "Pipe Network Diagnosis and Evaluation System (P-DES)," which has already been put into operation. For distribution reservoirs and other key facilities, on the other hand, Kobe City has constructed the "Water Supply System Reliability Assessment Program," which evaluates the reliability of the entire waterworks system and sets the renewal priority level of each facility. At the same time, the city is conducting diagnosis and evaluation of the earthquake resistance and degradation level of one facility after another in order to determine their current condition.

This paper reports on the reasonable approach that has been adopted in the series of programs for seismic assessment of existing distribution reservoirs.

2. SEISMIC ASSESSMENT PROCEDURES

In Kobe City, water is supplied to a population of approximately 1.53 million (roughly 670,000 households) from its 251 distribution reservoirs at 123 sites. Due to the sheer number of distribution reservoirs, it would be impractical to perform structural analysis and earthquake resistance diagnosis for each individual reservoir in order to understand their present condition, as this would incur a vast expenditure of time and money. Furthermore, should it be found that many of those facilities demand major seismic reinforcement work, a large amount of expenditures would then become necessary, thus hindering the smooth promotion of the project.

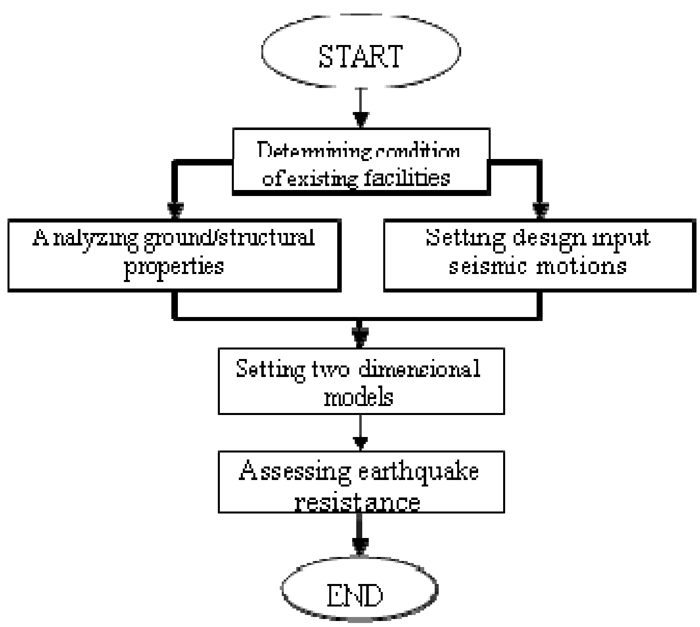


Figure 2-1

Therefore, Kobe City has taken notice of certain parameters which are thought to have a major impact on the earthquake resistance of distribution reservoirs, such as ground and structural properties. Then, the facilities were classified into groups of common characteristics, a detailed analysis was made of facilities representative of each group, and the earthquake performance of all of the facilities in each group was assessed based on the analysis findings. In order to reproduce with greater accuracy the impact that an earthquake would have on the facilities, this rational approach also included the setting of design input seismic motions and appropriate evaluation of the three-dimensional effect in such a way that matches reality. The work flow is shown in Fig. 2-1.

3. DETERMINING THE CONDITION OF EXISTING FACILITIES

The majority of Kobe City's distribution reservoirs are made of reinforced concrete (RC), followed in descending order by pre-stressed concrete (PC), steel (ST), and tunnel-type. These structural types were chosen on account of topographical restrictions, land restrictions, historical backgrounds, etc. (Fig. 3-1)

Structurally, the distribution reservoirs of Kobe City have much in common, since the great majority of them were built according to the city's original design standards which were established many years ago. This applies especially to those reservoirs that have been built since the time around 1965, when clear seismic criteria were laid down, and therefore they have produced some of the most useful data for seismic assessment.

If the design standards that were used for each structural type are considered, most of the PC and ST distribution reservoirs were designed according to nearly the same standards since their construction was concentrated during a certain period of time, which makes it easy to observe the tendencies in their earthquake performance. Meanwhile, RC distribution reservoirs, whose number is the largest, offered a great deal of interesting data for seismic assessment, including information concerning historical changes in design standards which span more than a century since the early days, variations of their types (aboveground vs. underground), and differences in their scale.

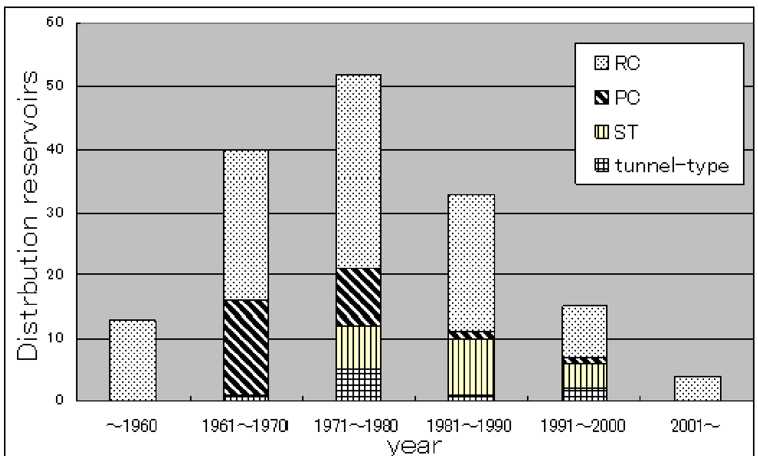


Figure 3-1

4. SETTING OF DESIGN INPUT SEISMIC MOTIONS

For the purposes of this earthquake resistance diagnosis, the static analysis technique for seismic assessment was used by setting a horizontal seismic coefficient based on acceleration response spectra. With this technique, seismic assessment is heavily affected by the dynamic characteristics(natural period, damping coefficient) of structures during an earthquake. Because of this, the following considerations were taken into account in order to accurately conduct diagnosis in such a way that matched the ground and structural properties of distribution reservoirs in Kobe. Below is an account of such considerations in the case of RC distribution reservoirs (aboveground type, the most common structure).

4-1. Preparation of Base Ground Surface Input Waveforms

For earthquake response analysis, it is a common practice to adjust the actual values of strong-motion earthquake records to generate waveforms that share similar characteristics with response spectra for earthquake-resistant design and use them as earthquake motions. For this study, NS component records observed at Kobe University during the Great Hanshin Earthquake of 1995 (Fig. 4-1) were used to generate earthquake motions for dynamic analysis. In more concrete terms, the amplifying characteristics of strong-motion earthquake records were varied within the frequency domain, so that they achieved characteristics close to the upper limit (90% non-exceedance probability) of base ground surface earthquake motions' velocity response spectrum*1 provided in the Earthquake-resistant Construction Method Guidelines for Water Supply Facilities (1997)[1]. This is because it was assumed that these observed waveforms would accurately reflect the ground properties of Kobe City's distribution reservoirs, which stand on relatively solid ground, as they were observed during an earthquake on firm ground with a thin sedimentary layer.

*1 The Earthquake-resistant Construction Method Guidelines for Water Supply Facilities (1997) use five earthquake engineering records observed in seismic bedrock during the Great Hanshin Earthquake of 1995 (NS・EW at Kobe University, GL-33m・N78E・N12W at Higashi-Kobe Bridge, GL-83m・N12E at Port Island) to define design response spectra.

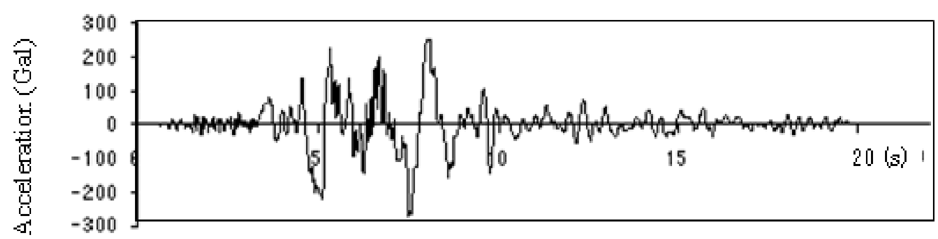


Figure 4-1 NS component records at Kobe University

By adjusting the amplitude of the accelerogram of Kobe University's NS component records (Fig. 4-1), the base ground surface input accelerogram in Fig. 4-2 was created

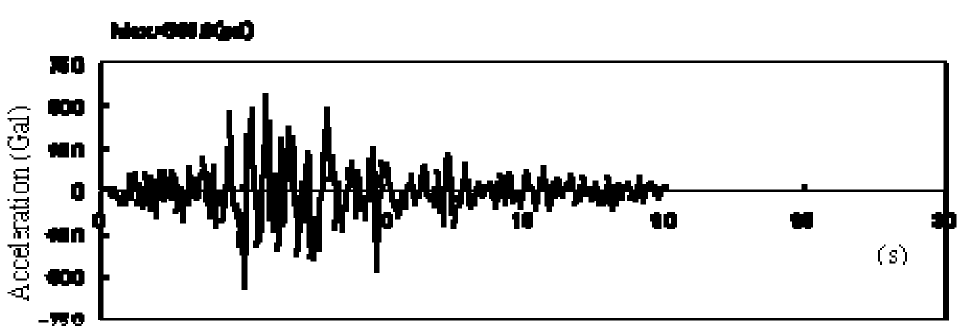


Figure 4-2 Base ground surface input acceleration wave for dynamic analysis

When input seismic motions on the base ground surface were created, an accelerogram was achieved which showed a maximum acceleration of 566.9 gal on the base ground surface. This maximum acceleration was more than twice that of the original waveform, since the waveform was deformed and amplified to attain characteristics similar to those of the velocity response spectra provided in the Earthquake-resistant Construction Method Guidelines for Water Supply Facilities (1997). However, this is below 818 gal, the maximum acceleration recorded on the ground surface during the Great Hanshin Earthquake. This is attributable to the fact that the distribution reservoirs were built on firm ground near the base ground surface.

4-2. Dynamic Characteristics of Structures

A typical RC distribution reservoir structure features low height, horizontal expansion, and high rigidity. Because of this, their natural period tends to be very short. Since their flat structure increases the ground contact area, the underground dissipation damping effect is expected to be large. Of the several methods for calculating a natural period, one was used which makes the calculation by analyzing the natural value of a dynamic analysis model, as it is capable of simultaneously calculating both the natural period and the damping coefficient. The analytical model (Fig. 4-3) features the scale, structure, and ground conditions which are most standard in Kobe City.

[Analysis conditions]
- Foundation structure: Spread foundation
- Foundation ground: N value; 50
- Relaid soil: N value; 10 (sandy soil)
- Inside water level: High water level
- Elements: See figure at right

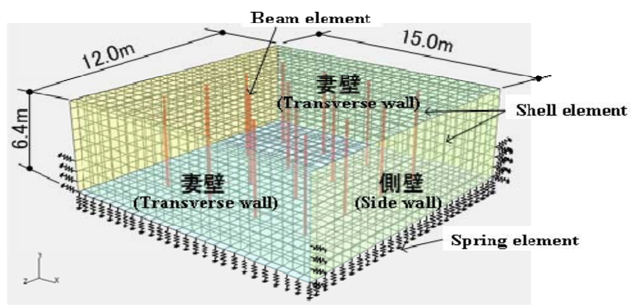


Figure 4-3

Table 4-1 Input physical properties of the building (shell element)

	Deck slab	Upper slab	Side wall	Transverse wall
Plate thickness t (mm)	550	300	550	550
Young's modulus E (kN/m^2)	2.500E+07	2.500E+07	2.500E+07	2.500E+07
Poisson's ratio ν	0.2	0.2	0.2	0.2

Table 4-2 Input physical properties of the center pillar (beam element)

Young's modulus E (kN/m^2)	2.500E+07
Cross section A (m^2)	0.25
Torsional constant J (m^4)	8.788E-03
Geometrical moment of inertia Iy = Iz (m^4)	5.208E-03

Table 4-3 Input physical properties of the ground (spring element)

		Deck slab	Side wall	Transverse wall (end side)
Coefficient of subgrade reaction*2 (kN/m^3)	X direction	29404	17713	5111
	Y direction	102912	5061	5111
	Z direction	29404	5061	17888

*2 Calculated using the method to determine natural periods for inclusion in the Specifications for Highway Bridges

For this study, analysis was also conducted for cases where distribution reservoirs are covered with fill soil (many such reservoirs can be found in Kobe City) to assess the impact of this factor.

The results of the analysis are shown in Tables 4-4 and 4-5. For distribution reservoirs covered with fill soil, both the natural period and the damping coefficient were slightly higher than otherwise, but were not high enough to have a major impact on diagnosis results. Also, for facilities aboveground whose N value is 50 or greater, their natural period is roughly estimated as below 0.15 (s), with a damping coefficient, around 15%. For reference, the Earthquake-resistant Construction Method Guidelines for Water Supply Facilities (1997) use 5% for the damping coefficient.

Table 4-4

		Natural period (s)	
		With fill soil	Without fill soil
Vibration direction	X direction	0.1398	0.1272
	Y direction	0.1576	0.1394

Table 4-5

		Damping coefficient (%)	
		With fill soil	Without fill soil
Vibration direction	X direction	15.4	14.8
	Y direction	14.8	14.5

4-3. Earthquake Response Analysis

Using the base ground surface input waveforms created from the aforementioned results, response analysis was conducted by assuming a damping coefficient of RC distribution reservoirs (aboveground type) of 15%. As examples of analysis, accounts are provided below for the two cases of a model on the base ground surface of N = 50 or greater and a model on the sedimentary layer over the base ground surface (a 6m-thick diluvial sedimentary layer <N = 30> on the base ground surface).

Shown in Fig. 4-4 are the results of response analysis. As the results show, the spectrum value for this analysis was smaller than that used in the popular Earthquake-resistant Construction Method Guidelines for Water Supply Facilities (1997) (damping factor: 5%), since the damping factor was assumed to be 15%. Response analysis of the model on the base ground surface and that on the sedimentary layer showed that the response on the sedimentary layer was slightly larger than that on the base ground surface. Thus, it was decided that an acceleration response spectrum on the sedimentary layer (bigger than on the base ground surface) be used as a spectrum for seismic assessment of this diagnosis.

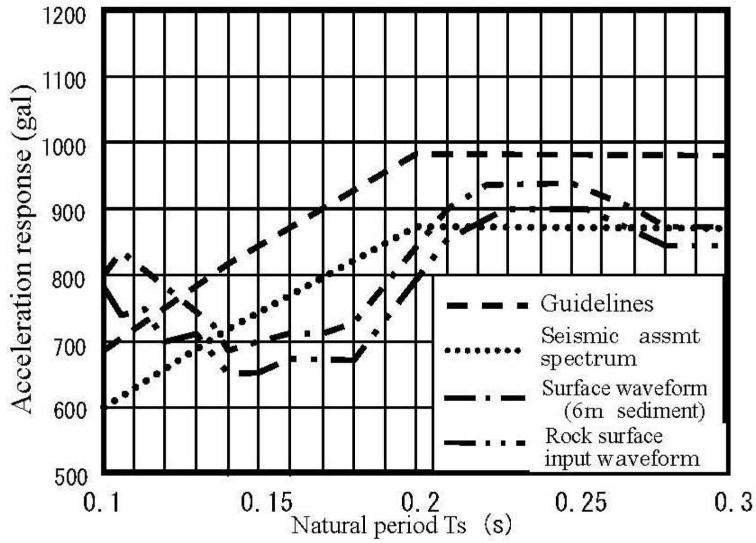


Figure 4-4

5. ANALYSIS OF GROUND/STRUCTURAL PROPERTIES

In order to conduct earthquake resistance diagnosis for a large number of distribution reservoirs both rationally and accurately, the reservoirs were classified into groups of reservoirs that have several characteristics in common. Discussed in this section are the findings of trend analysis conducted on characteristic values of RC distribution reservoirs.

As in the previous section, a three-dimensional model of a flat slab structure, the most standard type of RC distribution reservoir (Fig. 4-3), was used as a basic case model for this analysis. Parameters for this trend analysis were water level, N value of the ground, presence or absence of fill soil, presence or absence of unsymmetrical earth pressure, and presence or absence of embedment, which were varied to assess their impact on sectional force during an earthquake. For the purposes of this study, the load due to seismic force was calculated using the horizontal seismic coefficient Kh = 0.33.

In more concrete terms, the impact on earthquake performance was assessed by varying each parameter in Table 5-1 and comparing the analysis results of each case.

From these analysis results, it appears that each parameter has the following impact on sectional force:

[Cases 1 and 2: Water level]

The water level of the reservoir had little impact, but affected the axial force of the deck slab, upper slab, and side wall, and the bending moment and shear force of the side wall.

[Cases 3 and 4: N value of bottom ground]

No different from the basic case at an N value of 30, but a major difference was observed in the deck slab and the lower part of the side wall at an N value of 10.

[Case 5: Fill soil]

No clear difference was observed which demands grouping, but some impact was observed on the bending moment of the upper slab and deck slab and the axial force of the deck slab and side wall.

[Case 6: Unsymmetrical earth pressure]

When unsymmetrical earth pressure is present, the axial force and bending moment of the deck slab and upper slab, and the bending moment and shear force of the side wall, were more affected when compared to the basic case.

[Case 7: Embedment]

The impact on sectional force was found to be relatively large. When setting is not present, the side wall's compressive axial load is lower but its bending moment is greater than otherwise.

[Cases 8, 9 and 10: Dimension in planning]

As the dimension in planning (L/B) increases, the effect of the traverse wall decreases, until it became almost nonexistent at L/B = 3.0.

These results were used as parameters by which distribution reservoirs were grouped according to their structural characteristics, and as parameters for an estimated formula for generating sectional force, which is discussed under "Seismic Assessment" in a later section.

Table 5-1

Analysis cases	Parameters	
Case 1	Water level	Design water level
Case 2		Operational water level
Case 3	N value of bottom ground	N=10
Case 4		N=30
Case 5	Fill soil	Yes / No
Case 6	Unsymmetrical earth pressure	Yes / No
Case 7	Embedment	Yes / No
Case 8	Dimensions in planning ※Ratio of the space between transverse walls (L) to the width of transverse walls (B)	L/B=1.4
Case 9		L/B=2.2
Case 10		L/B=3.0

6. SETTING A TWO-DIMENSIONAL MODEL

For the purposes of this seismic assessment, a two-dimensional analysis model was adopted. In order to reproduce the effect of transverse walls which serve the same function against earthquake load as earthquake resisting walls, an analysis model was set where thin-walled plate elements were arranged on the cross section of a two-dimensional model. (See Fig. 6-1)

In order to create a two-dimensional model that is consistent with the analysis results of the three-dimensional model, the following considerations were taken into account: Changes in rigidity due to differences in the aspect ratio of the dimensions in planning were reproduced by varying the rigidity of thin-walled plate elements within the rigid frame, and changes in the space between and thickness of side walls were reproduced by varying the plate thickness of thin-walled plate elements.

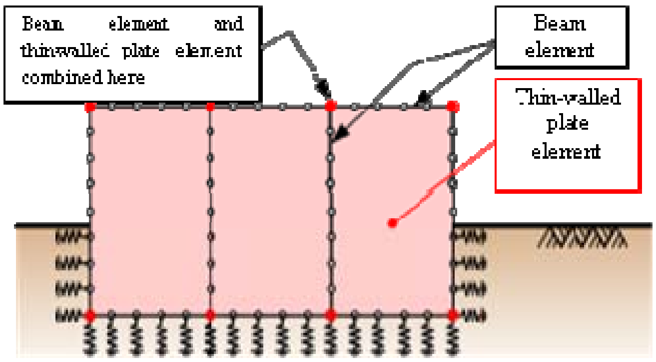


Figure 6-1

Prior examination showed a tendency for seismic assessment of distribution reservoirs to be harshest for the bending moment of the deck slab and upper slab. Because of this, models which reproduce the sectional force of the deck slab and upper slab during an earthquake were used for this analysis. Shown below are the models used (Figs. 6-2 and 6-3) and the analysis conditions (Tables 6-1, 6-2, and 6-3).

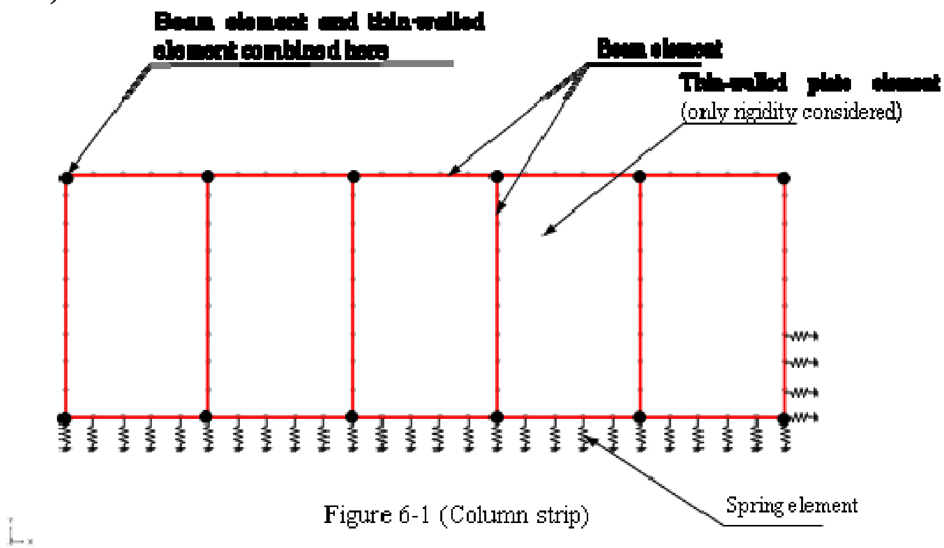


Figure 6-1 (Column strip)

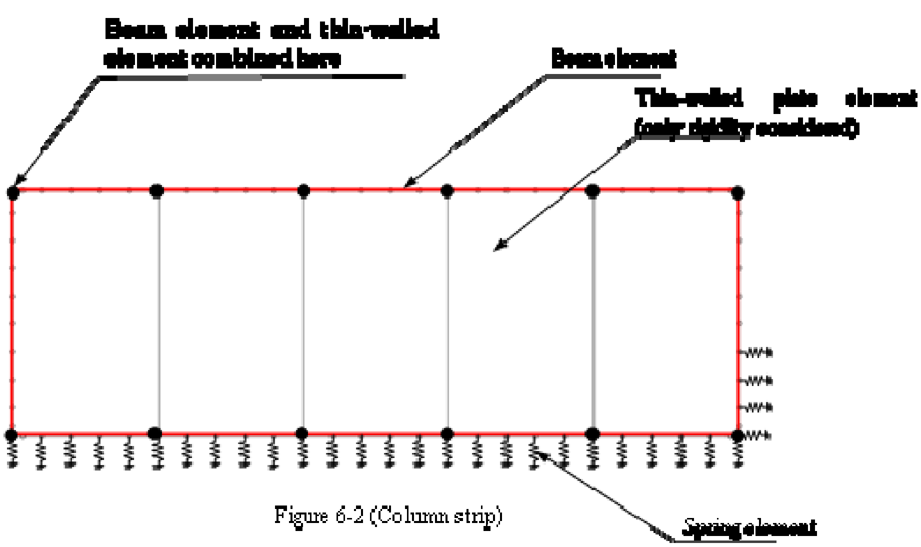


Figure 6-2 (Column strip)

Table 6-1 Cases for analysis

No.	Physical properties of thin-walled plate elements		Remarks
	Young's modulus E_2	Plate thickness	
1	E_c^{*}	t_{eq}^{**}	
2	$0.5 \cdot E_c$	t_{eq}	
3	$0.2 \cdot E_c$	t_{eq}	
4	$0.1 \cdot E_c$	t_{eq}	
5	$0.05 \cdot E_c$	t_{eq}	
6	$0.02 \cdot E_c$	t_{eq}	
7	0	0	No thin-walled plate element

Here, Ec (Young's modulus for the building's concrete) = 2.5×10^7kN/m^2, and
t_{eq} (equivalent plate thickness of the transverse wall) is calculated using the following equation:

$$t_{eq} = \frac{2\times t_w}{B} = \frac{2\times 0.55}{18.8} = 0.0585 \ \text{(m)}$$

where t_w (thickness of the transverse wall) = 0.55 (m), and
B (space between transverse walls, or depth of the structure) = 18.8 (m).

Table 6-2 Conditions of the structures

	Deck slab	Upper slab	Sidewall	Transverse wall	Center pillar
Geometrics (mm)	Plate thickness t = 550	Plate thickness t = 300	Plate thickness t = 550	Plate thickness t = 550	500×500 ctc3650
Young's modulus E (kN/m^2)	2.500E+07	2.500E+07	2.500E+07	2.500E+07	2.500E+07
Unit weight γ (kN/m^3)	24.5	24.5	24.5	24.5	24.5
Poisson's ratio ν	0.2	0.2	0.2	0.2	0.2

Table 6-3 Coefficient of subgrade reaction (kN/m^3)

		Lower part of deck slab	Sidewall	Transverse wall (end side)
Under stationary load	X direction	5758	——	——
	Y direction	20153	——	——
	Z direction	5758	——	——
Under incremental load during earthquake	X direction	11516	17957	5181
	Y direction	40307	5131	5181
	Z direction	11516	5131	18135

The analysis discovered that, among the analysis models listed above, Case No. 1 in Table 6-1 demonstrated the most reproducibility. With the existing distribution reservoirs, however, the fact that the dimensions in planning (ratio of the space between the transverse walls to width of the transverse wall, or aspect ratio L/B) vary greatly would have a significant impact on results. As such, three-dimensional models were used to analyze four cases which used aspect ratio as a parameter (L/B: 1.0, 1.4, 2.2, 3.0), so as to find the optimal value of the Young's modulus of the thin-walled plate element (equivalent Young's modulus). The analysis findings are shown in Fig. 6-3.

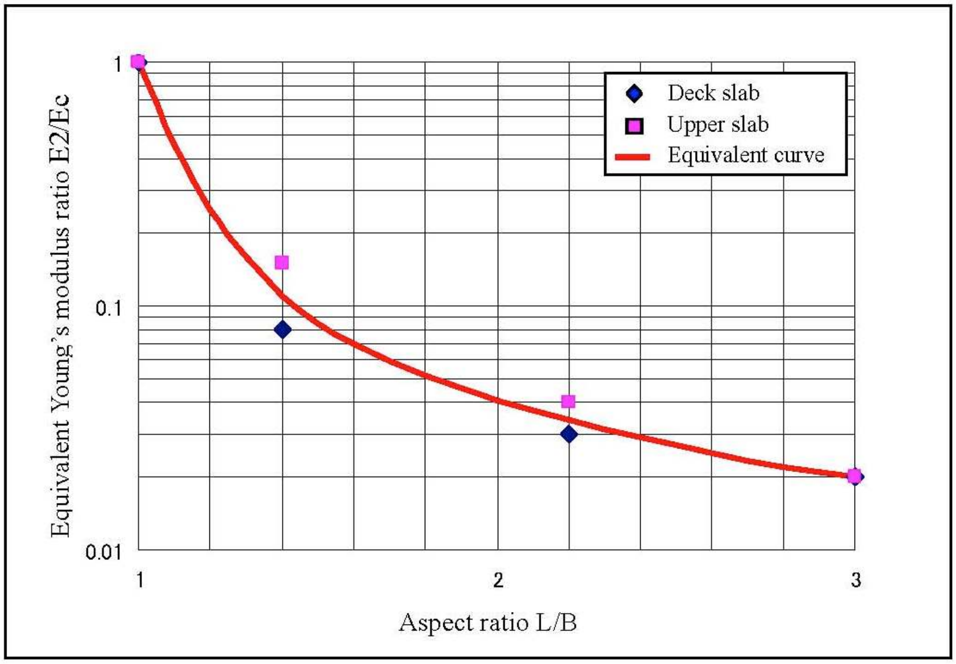


Figure 6-3

For this seismic assessment, two-dimensional models were used after being made consistent with the analysis results of three-dimensional analysis models using a different thin-walled plate element for each distribution reservoir, as above.

7. SEISMIC ASSESSMENT

This seismic assessment is characterized by the fact that, instead of making individual assessment, only the facilities which represent each group of structurally similar facilities were subjected to structural analysis, and that all of the facilities in each group were assessed based on the findings of such structural analysis. For this purpose, the results of previous analysis were used to create a simple estimated formula to calculate the bending moment and shear force, and then a simple estimated formula was created to calculate the flexural strength and shear strength for comparison. Then, a graph was created in order to determine the relationship between the calculated sectional force and strength, and assessment was made accordingly. An example of such a graph is shown in Fig. 7-1.

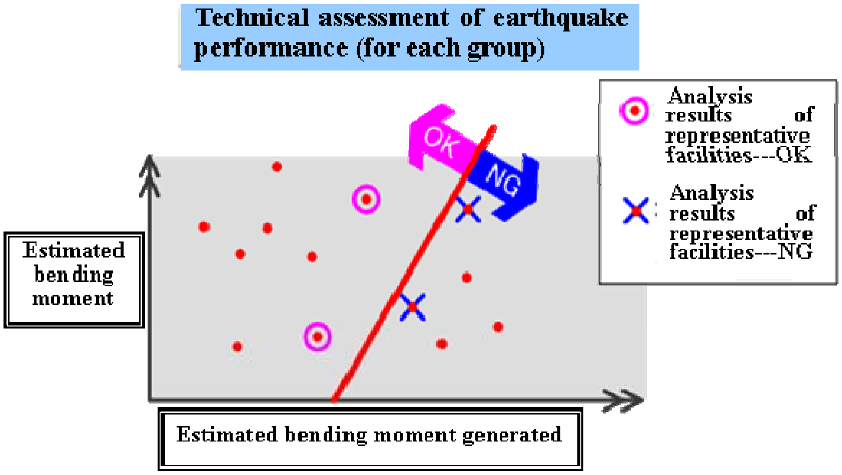


Figure7-1

8. CONCLUSION

A project to make existing facilities earthquake-resistant is easily subject to financial restrictions, since it does not make any direct contribution toward increasing the earnings of the water supply business. Partly because of this, such projects may not proceed as smoothly as hoped. Meanwhile, the results of attempts to assess the earthquake performance of such facilities can vary depending on which analysis techniques are used. For instance, there is a possibility that the need for anti-seismic reinforcement may be significantly reduced by using a technique other than that used in this report, such as detailed investigation of damage evolution by appropriately assessing nonlinearity. Because of this, it is important to incorporate various innovations to conduct seismic assessment that is both accurate and appropriate. We hope that the reasonable assessment technique reported here will be useful for operators of the water supply business, who own a large number of water supply facilities, to drive their projects to make facilities earthquake-resistant.

The authors wish to express their deepest gratitude to everyone who offered their assistance in conducting data analysis and structural analysis for the writing of this report.

REFERENCES

[1] Earthquake-resistant Construction Method Guidelines for Water Supply Facilities (1997), Japan Water Works Association

[2] Introduction of a Simplified and Rational Earthquake Resistance Diagnosis Technique for Existing Distribution Reservoirs (2007), Kobe City Government

[3] Example of a Rational Earthquake Resistance Diagnosis Technique for Existing Distribution Reservoirs (2007), Nihon Suido Consultants Co., Ltd. and Kobe City Government

Influence of Organizational Structure on Effectiveness of Post-Disaster Management

Jiin-Song Tsai[1] and Chieh-Chi Cheng[2]

ABSTRACT

In the wake of a devastating disaster, the suddenly induced demands from all aspects could immediately hold down an administrative organization. In order to deal with such inextricable challenge, employing an ad-hoc organization to temperately take in charge is a pragmatic strategy. This paper presents a real practice of such strategy for post-disaster mitigation in 1999, when an earthquake stroke Taiwan causing serious damages including knocking down a high-rise apartment building in Taipei. With the delineation of a retrospective investigation, it presents a comparative study of the effectiveness of organizations in two different structures (a flat structure and a hierarchical structure) upon a four-phase work of the post-disaster mitigation (i.e. rescuing victims, demolishing wrecks, restoration of community utilities, and reconstruction). The in-situ situation is retraced via interviews with witnesses and participants. Data recovered from those interviews as well as extant documentation are used to develop numerical models for extensive simulations employing VDT, i.e. a computational code for organization analysis. Our results show that organization in flat structure is an effective practice to deal with such emergent case when timing is on the top list. It allows bypassing cumbersome barricades of coordination in a hierarchical organization in the early two phases; however the according benefits soon vanish in the latter two phases. Our findings conclude that a flat-structure ad-hoc organization for handling emergency should enlist people who are direct involved in the administration system to cut out huge coordination burdens. On the other hand, the benefit of employing such organization is not very obvious for the latter restoration and reconstruction works when the emergency is eased.

INTRODUCTION

The crisis of a sudden disaster can overwhelm a normal administrative organization in a surprisingly short time when all its functions designed for regular purposes are disengaged from their normal employment and impromptu adapted to disaster-recovery activities. Despite well documented disaster plan in hand, serious management problems regarding the way of communication and the authority of making in-time decision still easily break out due to the lacking of coordination under such an extreme chaos [1, 2]. Typical case of natural disaster, such as a devastating seismic event, public rescue/fire services would be overtaken by urgent requests when the situation remains vague and turbulent and their supportive backups are unready.

Due to lack of consensus among people involved, enormous difficulties in achieving an overall coordination first emerge from the ambiguity of higher echelon personnel to exercise authority to resolve conflicts over numerous concurrent tasks, clashes over jurisdictional differences, and disputes over over-works. The interruption of regular communication channels in between further worsens the case.

[1] Jiin-Song Tsai, Professor, Dept. Civil Engineering, National Cheng Kung University, Tainan 70101, Taiwan, R.O.C.
[2] Chieh-Chi Cheng, Ph.D. Student, Dept. of Civil Engineering, National Cheng Kung University, Tainan 70101, Taiwan, R.O.C

It happened once; when a magnitude 8.1 earthquake struck Mexico on September 19, 1985 and a breakdown government was incapable to quickly response and take actions, leaving citizens to organize rescue efforts themselves, against this major scale tragedy [3]. The Mexican government revealed its incompetence in failing to act decisively in coordinating rescue efforts that caused over ten thousands lives as the consequence. In the days immediately following the earthquake, the central government, the army, the police, and the local government of Mexico City appeared to operate without direction and coordination; and the police and the army even hindered the progress of citizen rescue efforts by cordoning off damaged buildings. An overall criticism concluded that the Mexican government was unresponsive, incompetent and irresponsible in the 1985 earthquake [4, 5]. Many authorities including the country president Miguel de La Madrid confessed that their governmental meltdown was mainly due to "lack of coordination." Their failure was not an exception since another breakdown government shown in Japan when a magnitude 7.2 earthquake smashed the Kobe-Osaka area on January 17, 1995. The malfunction of the governmental organization held down crucial in-time rescues until the third day since the calamity and left an unrecoverable regret for losing more than five thousands lives [6].

Many more tragedies were inevitably cursed with poor emergency management alike except once when another massive earthquake (magnitude 7.1) shook Taiwan on September 21, 1999 around midnight causing a critical situation in the capital city Taipei. In the dark turbulent moment, a 12-story apartment building was reported totally knocked down while wrecked another building next to it (Figure 1). Instead of hanging in there for a higher prefect's command, the governmental officers first attended the site promptly set up an in situ headquarters to in charge of commanding and coordinating all collaborative actions among numerous rescuing teams from domestic and international. Avoiding confusion from gigantic governmental hierarchy, the ad-hoc organization (headquarters) effectively performed their works on not only circulating important information regarding needs for additional efforts and resources but cultivating an impressive achievement of saving 107 lives from the remains of the quake-battered building within the first 24-hour and miraculously dug out another 2 survivors under the rubble on the sixth day [7].

Figure 1. Collapse of an apartment building in the 1999 Taiwan Earthquake [7]

Facing major events, most rescuing actions respond ahead of clear understanding of the magnitude of the disaster. Units and individuals rush to the site with resources to seek places for use, and all actions usually begin in an unorganized manner. Each action may be isolated from others and have to work alone. At this moment in time, key issue is to organize these people and actions and point out obvious needs for their bring resources. Earlier studies suggest that at the conjuncture of the two floods of information, resources availability and in-time needs, a capable "headquarters" is the keystone to overpass onerous tasks against the crisis (e.g. [1, 2, 8, 9]). In the Taipei case, the in-situ headquarters was accordingly established to organize people coming to the site and to coordinate all the emergency activities once the awareness of needs and resources coincide at the site. This ad-hoc organization emerged and mainly formed by governmental officers, and their power rested not only on formal authorization but upon superior capacity to coordinate via a special structured organization.

Thompson [9] emphasizes that the structure of an organization is the fundamental vehicle by which the participating members could sensibly control over resources, issue commands, delimitate authorizations and responsibilities to achieve their goals in expectation. What the "structure" mentioned herein is the internal differentiation and linkages of the members in the organization; that is a social system containing both human and nonhuman resources.

Our findings in a preliminary study of the Taipei case [10] reveal that the effectiveness of management shown in the phenomena of coordinated actions under an unexpected situation is greatly dependent on the structure of organization. It is then an attempt to experiment on organizational structure to delineate instrumental reasons that account for the effectiveness of post-disaster management. We bring this attempt into numerical simulation employing a computational model of project organizations, the Virtual Design Team (VDT) [11], to analyze how organizational structure may affect the coordination capacity of organization, with resulting impacts on the effectiveness of management.

The VDT model is built based on organizational contingency theory [12] according to our exploitation about collaborative and multidisciplinary actions for the post-disaster mitigation of the Taipei case. For given scenarios and organization setting, VDT generates the corresponding emergent organizational performances through simulating behaviors of, and interactions among, the participating members. In the present study, the obtained results are carefully examined over a three-way qualitative consistency among predictions of the numerical simulations, of organizational theories, and of experienced participants of the case.

Previous experiences indicate that the recovery from a natural disaster is a protracted process that takes immense efforts to proceed. Many consecutive works starting from the urgent situation are indispensable for healing the wounds. Taking the Taipei case for instance, these works is simply categorized into four phases: rescuing victims, demolishing wrecks, restoration of utilities, and reconstruction. Since Burns and Stalker [13] point out that an organization tends to perform differently under various environmental conditions, the identified four phases offer a series of "conditions" for analyzing the effectiveness of organizations throughout the entire post-disaster process.

As mentioned in above that different approaches to structuring organizations would result in differential effectiveness, much work herein is done attempting to compare an ad-hoc organization (the in-situ headquarters) and a function organization (the municipal government), i.e. a flat structure and a hierarchical structure, respectively, upon the execution of post-disaster tasks in which their effectiveness is evaluated over the designated four phases. Since the structure affords numerous aspects to facilitate the coordinated actions of all kinds of interdependent tasks, it appears that we need to depict the effectiveness of an organization in respect of its coordination capacity toward handling the vast coordination requests in the following.

REQUIREMENTS VS CPACITY OVER COORDINATION

Organizational effectiveness is about how effective an organization is in achieving the outcomes it intends to produce. An effectively functioning organization integrates coordinated actions to produce wishful outcomes by its members who may need to compete in between for resources and their priority quite often. As a matter of fact, the overall capability to handle joint actions and resolve possible competitions (or conflicts) is so called coordination, and NSF-IRIS [14] well puts it: "the joint efforts of independent communicating actors towards mutually defined goals." NSF-IRIS further depicts these efforts: "the emergent behavior of collections of individuals whose actions are based on complex decision processes." Malone and Crowston [15] broaden this seemly indescribable notion the act of working together, and they doubt people would appreciate good coordination unless lacking of it, like what Mexican president revealed in the 1985 Mexico earthquake event.

In an effective organization, all the matters have to do with coordination: dividing overall goals into a series of tasks (or actions), assigning these tasks to groups or to individual actors, allocating resources among the associated activities, and the last but not the least, sharing information among the actors to achieve the overall goals. In this regards, good coordination proceeds with well information processing to do things and solve problems. Malone [16] even argued coordination is: "the additional information processing performed when multiple, connected actors pursue goals that a single actor pursuing the same goals would not perform."

Earlier than NSF-IRIS [14] stated coordination "information processing within a system of communicating entities with distinct information states," Tushman and Nadler [17] has taken the view of an organization (a set of subunits) as an information processing system facing uncertainty and indicated that the greater the uncertainty, the greater are the information processing requirements for the whole organizational structure. They extend this concept for the design of organizational structure and stress that properly designed structures would bring up wanted capacities for effective information processing. From this idea, the basic issue is to constitute an assured information-processing capacity against the demands of the required works. To a certain extent, achieving a fit or match between the information-processing capacity of the organization's structure and the corresponding requirements is the practice to be effectiveness [12, 18, 19]. Figure 2 depicts a conceptualizing process of such practice for designing an effective organization.

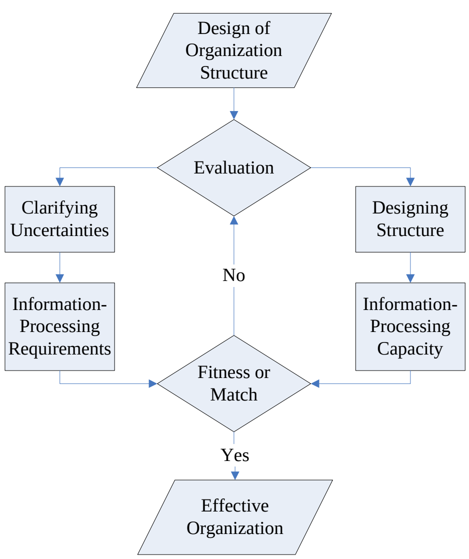


Figure 2. Concept of designing effective organization.

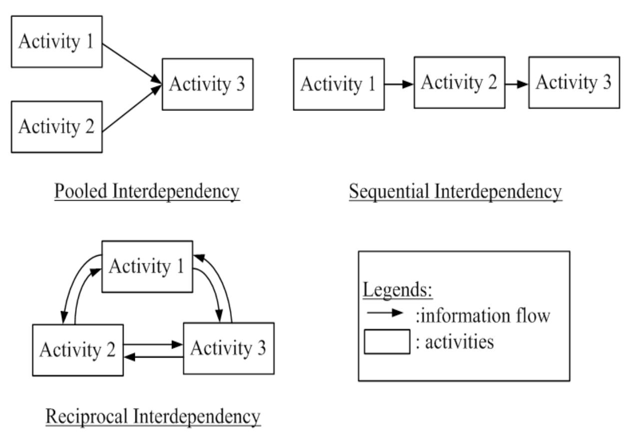


Figure 3. Interdependencies in between activities.

Organization has to deal with coordination problems mostly due to work-related uncertainty because of the complexity of each activity and the interdependencies in between different tasks (activities) [12]. In the literature, various task interdependencies among activities have been pointed out as crucial sources of such uncertainty [8]. Thompson [9] sums them up as three kinds of interdependencies: pooled, sequential and reciprocal dependency relationships representing the amount of associated coordination work from low to high. Shown in Figure 3, pooled interdependency represents a situation that each activity is a self-contained segment but renders a discrete contribution to the final result. Unless each performs adequately, the total accomplishment is threatened; failure of any one can jeopardize the whole work. Sequential interdependency represents the relationship of a serial form, in which a precedent activity produces things as inputs for the following one. To the whole work, the both make contributions and direct dependency can be pinpointed and specified in between them. The third kind is reciprocal interdependency, referring to the situation in which the outputs of each are inputs of the others. The distinguishing aspect of this kind is that each activity poses contingency for any other.

In the present study, the activities in the designated four phases to recover the Taipei disaster are of different independencies. For instance, in the earlier phases, i.e. rescuing victims and demolishing wrecks, most works relate to others in reciprocal interdependency since they demand very intensive interactions in between. And dependency relationships of the works in the latter two phases, i.e. restoration of utilities and reconstruction, are mostly sequential interdependency since their procedures mostly follow a series process. Taking Tushman and Nadler's [17] view of organization as information processing system facing uncertainty, the greater the uncertainty due to intensive reciprocal interdependency among the works in the earlier two phases, the greater are the information processing requirements for then.

Facing particular requirements, to obtain the desired information processing capacity the essence of organizational design is to deal with work-related uncertainty employing an optimal structure to place all members (or subunits) along the commanding lines together with suitable coordination and control mechanisms to function successfully. Basically the first is structuring a particular set of organizational arrangements choosing from a feasible set of structural alternatives to layout its authority/responsibility configuration to most effectively deal with the information processing requirements. The second is creating coordination linkages across the commanding lines to enhance the desired capacity in lateral. Galbraith [12, 18] proposed a range of coordination and control mechanisms to achieve substantial information processing capacity. And his idea can be summed up into two complimentary approaches: when the requirements are in quantifiable nature (e.g. planning and scheduling) setting formal meetings as the coordination mechanism is amenable, while the requirements are less quantifiable (e.g. informal communication) enhancing lateral relations are appropriate.

In the following we delineate the chosen case by first briefing the background in a logically sensible manner and then depicting the setting of VDT simulations over the four post-disaster phases. In a follow-up discussion, we elaborate on the useful lessons learnt from this case with an afterward scrutiny of the analytical results.

GOAL OVER POST-DISASTER PHASES

As a 7.2 magnitude earthquake roared through central Taiwan at 01:47 a.m. on September 21, 1999, Taipei was hard hit despite its remote location from the epicenter (200 km away). In the wake of the earthquake, a fire brigade responding to an emergency request for assistance attained to a scene and soon comprehended that the extent was a massive devastation. The situation then turned into a crisis; when over one hundred entrapped people were buried under the debris of a totaled apartment building, water and gas mains were broken, and fire was burning out of control hindering the movement of rescue services. To effectively contain the crisis lay ahead for the devastated area, the deputy mayor and a few municipal officers who rushed to the scene set up an in situ headquarters to quickly take in charge.

As mentioned earlier, the headquarters' achievement was mainly due to successful coordination between all the participating members and activities employing a flat-structure organization, in which members bypassed many cumbersome barricades and performed their works much effectively under an extreme situation. Nevertheless, the post-disaster recovery is a protracted process covering many others beyond such urgent moment. According to our preliminary study [10], the process covers, at least, four phases: (1) rescuing victims, (2) demolishing wrecks, (3) restoration of utilities, and (4) reconstruction. It is then of our interest to examine the effectiveness of such organization throughout those consecutive phases when the applicable goals shift in between.

There are many ways to measure the effectiveness of an organization [19]. Many theoretical perspectives provide various indicators that can account for the diversity in usage of effectiveness measurements [20]. Among them, Scott's rational perspectives [21, 22] emphasize goal attainment and focus on indicators including efficiency, productivity, and quality as outcome variables. We adopt these indicators to conceptualize organizational effectiveness in post-disaster management. Simply put, we take Scott's organizational effectiveness indicators and map them onto the dependent variables in our analyses at the outcome levels. Under this framework, the success of attaining goals for an organization determines whether it is effective or not. For the Taipei case that covers four phases, the evaluation criteria need to change with the goals for different phases. In the following, background of the effectiveness evaluation is briefly depicted.

As with most major earthquakes, this community disaster precipitated a mass convergence of responders besides the municipal agencies upon the disaster site for rescuing victims (Phase 1). The headquarters integrated all participants' activities to accomplish a novel goal of saving life to handle the mass casualties. In this regard, efficiency (rapidity) was the top indicator (outcome variable) of the effectiveness then. Nevertheless, factors seriously dragging the rapidity of the emergency response were those main difficulties emerged from a lack of previous experience between groups in such a joint enterprise. Even local agencies accustomed to working together, such as police and fire departments, encountered a few bewilderments at the height of the crisis when everyone was eager to engage in. The common issue for all participants was to cope with exigencies of information flow by any mean to meet the requirements of the situation. Given this, how to initiate and effectively maintain formal information flow between officials unfamiliar with others was the upmost concern for such an ad-hoc organization, and this concern continued to Phase 2 (demolishing wrecks) when heavy equipments arrived at the site to carefully remove the totaled building in order to scrupulously dig out any possibly survivors entrapped beneath the rubble.

Experiences of rescue experts remind that entrapped people can be pulled alive from the debris, but their chances of survival shrink rapidly with each passing hour. Therefore rapidity was an effectiveness indicator without doubt for rescuing victims in the first two phases. In addition, digging out entrapped survivors under the remains in Phase 2 needed to coordinate many heavy equipments under a clear working process. This process was not only to safeguard all the workers working together at the site but to ensure all the building was searched and everyone was rescued. The importance of "process quality" emerged and became another effectiveness indicator then.

Besides the immediate response stated in above addressing life safety issue, relief and recovery for the disaster community returning to normal is the next concern following the catastrophic disasters. Indeed, restoration of utility systems (Phase 3) and reconstruction of the building (Phase 4) addressing the latter concern of post-disaster works emphasize the goals of "restoring the functions of the disaster-stricken area and housing the disaster victims to return normal lives" and "re-developing a new community," respectively. For the both phases, their final "products" are surely of the main concern, and the outcome quality ("project quality") is then the important indicator of effectiveness. In the meantime, they differ from each other since the restoration of utilities still ask for rapidity, while the reconstruction works address the process quality of administrative procedure that usually takes some time to achieve. The particular concerns distinguish the stage goals for different phases, and Table 1 below summarizes them and the main effectiveness indicators for each phase.

Table 1. Effectiveness indicators for post-disaster works.

Phase	1	2	3	4
Goal	Rescuing	Demolishing	Restoration	Reconstruction
Effectiveness Indicators	Rapidity	Rapidity & Process quality	Rapidity & Project quality	Process quality & Project quality

OPERATION POLICIES OF ORGANIZATIONS

Post-disaster works (immediate response as well as the following recovery) are complex and often involve many interdependent activities, and require intensive coordination among people to deal with the activity interdependencies. To elaborate the effectiveness (and efficiency) of such works, we needs to explain how coordination requirements are generated in organization and what operation policies (coordination mechanisms) can be applied for given situations.

As mentioned earlier, the fitness between the demand and the capacity of coordination can demonstrate the effectiveness. Facing the applicable goals shift in between consecutive phases, contingency theory suggests that organization should adapt to the situations to gain the corresponding fitness. Therefore, to achieve its expected capacity as well as effectiveness, an organization has to function properly in accordance with suitable operation policies regarding (1) the level of centralization for decision-making and (2) the level of formalization for information exchange and communication. Taking the organizational structure as a system, the former is the coordination in vertical, and the later is in horizontal. In this study, we attempt to present a comprehensive study of two organizations (a flat structured and a hierarchical structured ones) employing varying policies upon the execution of post-disaster works. Table 2 summarizes their policies obtained by our investigations in the preliminary study [10].

Table 2. Operation policies of the post-disaster works [10].

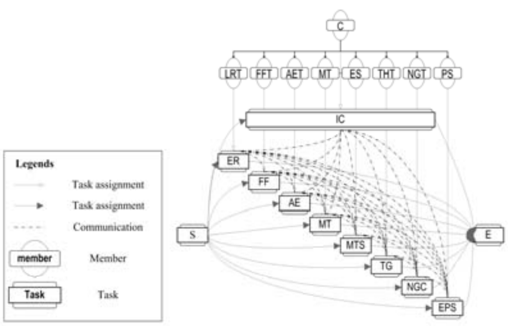
Phase	1 Rescuing	2 Demolishing	3 Restoration	4 Reconstruction
Ad-hoc organization (flat structure)				
Decision making	De-centralized	De-centralized	Semi-centralized	Centralized
Information exchange	Informal	Informal	Semi-formal	Formal
Functional organization (hierarchical structure)				
Decision making	Centralized	Centralized	Semi-centralized	De-centralized
Information exchange	Formal	Semi-formal	Semi-formal	Informal

It is worthy noted that the ad-hoc organization is the one started as the headquarters at the site to handle the earthquake induced issues, and the functional organization represents the municipal government that deals with routines mostly. Because of the fundamental difference, in the beginning, members of the ad-hoc organization make their own decisions and communicate with others for information exchange by any mean. Appearing an opposite attitude, those in the functional organization tend to hang in there for the decisions of higher prefect's command and to communicate and exchange information with others via formal channels and meetings. When their works shift to the later restoration and reconstruction, the ad-hoc organization operates in a very different way. Decisions are mostly made by the leader who has clear idea about requirements and quality of the final results, and formal meetings and written documents are employed to clarify confusions and deliver instructions. On the other hand, to the municipal government, the restoration and reconstruction are so similar to commonly seen construction/development cases that lower level officers are able to handle following a standard procedure. And these officers are accustomed to working together, most information exchanges in between are via informal ways.

COMPUTER SIMULATIONS

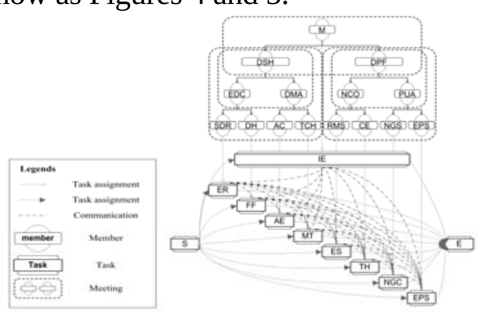
Computer simulation is commonly applicable to answer what-if questions in probing problems of organizations and is thus widely applied as a research method for organizational researchers [23, 24]. In some practices, computational approach can link up individual organization members to access organization behavior and performance collectively (whole organization) [25]. In this study, numerical experiments employing the VDT model are to answer the given research question: how organizational structure may affect the effectiveness of management upon the execution of post-disaster works.

Organization is structured to coordinate and control a specific set of tasks to achieve its goals. Information flows are lifeblood to an organization, and its function strives to create efficient information flows to be effective. Adopting Weber's [26] and many other organization theorists' [12, 27] consecutive works, VDT modeling method has made it representationally and computationally feasible to address human coordination issues through explicit representation of tasks, actor behavior and coordination actions. In a VDT model, organization is an information-processing system comprised of limited-capacity information processors representing its members (individuals or sub-teams). These information processors are organized in a structure with specific lines (lines of authority) denoting the supervision/report routes in between. Therefore, a VDT model is composed of three key conceptual components including (1) tasks generating information to be processed, (2) members processing and communicating information, and (3) a structure that constrains members' interaction behavior. Our VDT models of Phase 1 show as Figures 4 and 5.



Position	Description	Task	Description
C	Commander	IC	Infomation Control
LRT	Labor Rescue Team	ER	Life Rescuing
FFT	Fire Fighting Team	FF	Fire Fighting
AET	Ambalance Escorting Team	AE	Ambalance Escorting
MT	MedicalTeam	MT	Medical Treatment
ES	Equipment Supply	MTS	Equipment Supplying
THT	Technological Help Team	TG	Technological Helping
NGT	Natural Gas Team	NGC	Natural Gas Securing
PS	Power Supply	EPS	Electrical Power Supplying

Figure 4. Ad-hoc organization (flat structure) for Phase 1.



Position	Description	Task	Description
M	Mayor	IE	Information Exchange
DSH	Dept. of Security & Health	ER	Emergent Rescue
DPF	Dept. of Public Works & Finance	FF	Fire Fighting
EDC	Emergency Dispatch Center	AE	Ambalance Escorting
DMA	Division of Medical Affairs	MT	Medical Treatment
NCO	New Construction office	ES	Equipment Supplying
PUA	Public Utility Administration	TH	Technological Helping
SDR	Station of Disaster Rescue	NGC	Natural Gas Securing
DH	District HQs	EPS	Electrical Power Supplying
AC	Ambulance Center		
TCH	Taipei City Hospital		
RMS	Road Maintainance Section		
CE	Construction Engineers		
NGS	Natural Gas Section		
EPS	Electrical Power Section		

Figure 5. Functional organization (hierarchal) for Phase 1.

In the shown models, the task precedence model shown in the lower part displays all tasks for rescuing victims. To capture the coordination work load that would arise from this extremely concurrent schedule, we augment the workflow layout with two additional links, "communication links" and "rework links", depicting the interdependency between tasks. Those tasks of reciprocal dependency relationship (Fig. 3) are connected by communication links (broken lines between tasks), and need to be tightly coordinated when they run in parallel. Rework links between two tasks (not available in Phase 1 works) show their sequential dependency relationship (Fig. 3), and indicate that any significant exception—an unexpected change or error/mistake—in the precedent task will trigger compensating rework in the following one. In the upper part of the figures, all the tasks bond to (using solid lines) responsible members who are the processors to handle all the associated information flows to their assignments. The supervision/report relationships between these members are shown as the organizational structure, in which member icons represent their positions and the containing framework describes their reporting hierarchy.

In a complete VDT model, we captures the total effort that includes the direct work, plus all of the required communication work to coordinate interdependent tasks, the supervision work to answer request questions, and an allowance for the rework that will propagate between parallel tasks as changes or errors occur. Besides, a total effort analysis should cover the time and need for "meetings", and in Figure 5 we identify three kinds regularly scheduled meetings, and embrace those members who needed to attend these meetings in separate confining boxes.

Once the total effort for completing the works is elaborated, we enhance the modeling with the corresponding operation policies (Table 2), so that the organization's information processing capacity to execute the total effort can be properly represented. Therefore, we described the decision-making policies of ad-hoc organization for Phase 1 works employing a low value to Centralization (how high up in the organization decisions get made); a low value to Formalization (to what extent team members would wait for meetings to coordinate, versus initiate quick queries to one another). Also we describe the working condition of Phase 1 by assigning a high value for Interdependency (to what extent high reciprocal relationship existing in between all the designated tasks); a high value for information exchange (how often in term of a ranking scale showing information exchange rate between members); and a low value for Team Experience (the extent to which members of this team had previously worked together). Each of these parameters affected the micro-decision-making behavior of all the members of the organization, and so would have an impact on executing their assigned works. The parameter settings for all the other phases are delineated in Table 3.

Table 3. Parameter settings of operation policies for computer simulations.

Phase	1. Rescuing	2. Demolishing	3. Restoration	4. Reconstruction
Descriptions of working condition				
Level of reciprocal interdependency	High	Moderate	Low	Low
Information exchange	High	High	Moderate	Low
Team experience (Familiarity)	Low	Low	Moderate	High
Ad-hoc organization (flat structure)				
Centralization	Low	Low	Moderate	High
Formalization	Low	Low	Moderate	High
Functional organization (hierarchical structure)				
Centralization	High	High	Moderate	Low
Formalization	High	Moderate	Moderate	Low

RESULTS AND INTERPRETATIONS

It should be mentioned that Figure 4 shows a "baseline model" used to calibrate our analytical results against the documented records [10]. By baseline simulations, we carefully calibrate the durations of Phases 1 and 2, in 3 and 6 days, respectively. Table 4 lists all the durations predicted using computer simulations, in which the two baseline simulations (Phases 1 and 2 of ad-hoc organization) build important stepping stones for the credibility of the intellective simulations (predictions) presented in below.

Table 4. Durations of Post-Disaster Phases (in days)

Phase	1. Rescuing	2. Demolishing	3. Restoration	4. Reconstruction
Ad-hoc organization	3.0*	6.0*	11.3	441.0
Functional organization	7.0	13.3	13.0	427.4

*: baselines for calibrations.

Figures 6 to 10 show our simulation results, in which durations, coordination work volumes, rework volumes, process quality, and project quality are elaborated in comparisons. They are to illustrate the effectiveness indicators depicted in Table 1.

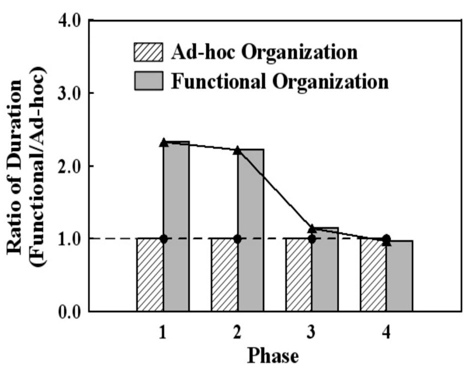


Figure 6. Ratios of durations (Function/Ad-hoc)

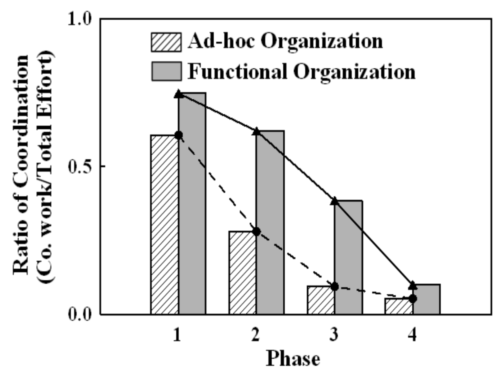


Figure 7. Ratios of coordination work volume (Coordination work/Total effort)

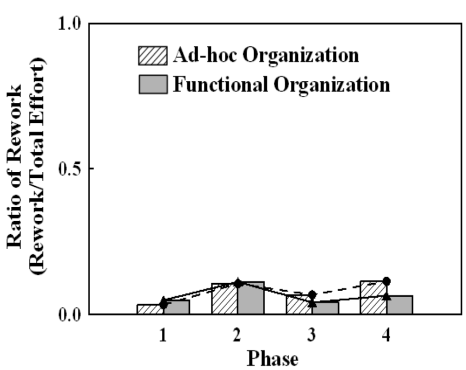


Figure 8. Ratios of rework volume (Rework/Total effort)

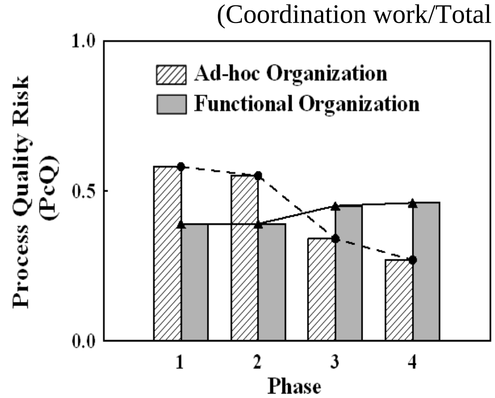


Figure 9. Process quality risk

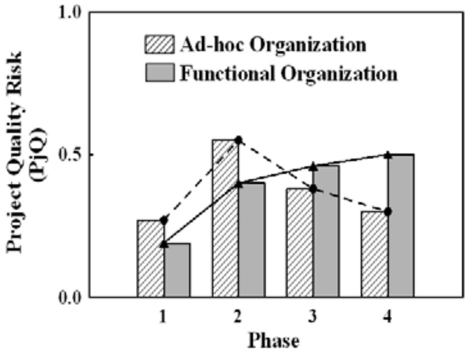


Figure 10. Project quality risk

As depicted in Table 4 and Figure 6, ad-hoc organization shows its superiority upon quick response immediately after disaster in phases 1 and 2. Shifting to the later two phases, the granted benefit vanishes, and functional organization demonstrates a similar efficiency for completion then. The results shown in the following figures deliberate our interpretations of this phenomenon.

In Figure 7, the results present a joint effect of situation (phase), operation policy (Table 2), and organizational structure. In particular, we can simply tell that coordination work is the main consumption of time (can be > 50% total effort) when highly reciprocal works emerges in Phases 1 and 2. In Phase 3, the shown results clearly craft the influence of structure when all the parameters of operation policy remain the same.

Figures 8 to 10 present the influence of operation policy, in which policies varying tightly relates with the tendency of shown results. This tendency is understandable once assuming that a high centralization having the leader to make decisions would enforce the will to correct any found errors/mistake and thus assure the quality of works at required standard. And a high level formalization employing formal meetings and written requests for information exchange would better the chance to clarify confusions and deliver instructions; so that errors/mistakes during the process would be taken care much thoroughly. Since the ad-hoc organization begins with a lower level of centralization as well as formalization and increase the level as the situation moves on toward the later phases; while the functional organization tends to function in opposite, the simulation results (Figure 8) demonstrate relatively less rework ratios in the first two phases for the ad-hoc organization as the consequence, but the tendency changes over in the later two phases. The same tendency is to the process quality risk (Figure 9) as well as project quality risk (Figure 10). It is worthy noted that the quality risk we employed herein is a scale to show a relative sense between the chosen two cases (organizations) rather than to indicate any sensible "risk." Details of the background regarding the calculation of the process risk and the project risk are well documented in a VDT manual [28].

CONCLUSIONS

This study is an attempt to answer how organizational structure may affect the coordination capacity of organization, with resulting impacts on the effectiveness of post-disaster management. This attempt leads to a numerical experiment employing the VDT model to exploit the influence of organization structure and associated factors on the mitigation actions for the 1999 earthquake disaster in Taipei. The main conclusions drew from our simulation results and findings are summarized as follows:

1. Post-disaster works (roughly including rescuing victims, demolishing wrecks, restoration of utilities, and reconstruction) are complex and often involve many interdependent activities, and

require intensive coordination among people to deal with the activity interdependencies.
2. An effective organization integrates coordinated actions so as to achieve wishful goals by its members who need to continually work out their priority of actions.
3. Ad-hoc organization is an effective practice to deal with emergent situations when functional organization is immobilized or overtaxed by a sudden natural disaster and rapidly rescuing victims is on the top list.
4. Employing flat organizational structure can effectively facilitate coordination for emergency bypassing cumbersome administrative barricades induced by hierarchical structure.
5. Ad-hoc organization for handling emergency should enlist people who are direct involved in the administration system to cut out huge coordination burdens.
6. The benefits employing ad-hoc organization are not obvious for dealing with later restoration and reconstruction works.
7. To achieve its expected effectiveness, an organization has to function properly in accordance with updated situations employing suitable operation policies regarding (1) the level of centralization for decision-making and (2) the level of formalization for information exchange.
8. Operation policies employing low levels of centralization and formalization are favorable to deal with highly reciprocal interdependent activities in the early stage of post-disaster when rapidity is the main concern of effectiveness.
9. Operation policies employing high levels of centralization and formalization are helpful to the quality of the resulted in works.

REFERENCES

[1] Quarantelli, E.L. 1988. "Disaster Crisis Management: A summary of research findings," *J. Manage. Stud.*, 25(4):373-384.

[2] Hy, R.J. and W.L. Waugh. 1990."The Function of Emergency Management," in *Handbook of Emergency Management: Programs and Policies Dealing with Major Hazards and Disaster*, W.L. Waugh and R.J. Hy eds., NY, Greenwood Press, pp. 11-26.

[3] The New York Times. 1985. Sep. 21, 22 & 25.

[4] Sánchez, V. 1986. *México un Pueblo en Pie: El Terremoto de 1985*. México, DF: Costa-Amic.

[5] Musacchio, H. 1995. *Ciudad Quebrada: Consequencias del Terremoto de 1985*. México, DF: Grupo Editorial Planteta de México.

[6] Kekoolani, D. 2000. *Great Quake: Kobe, Japan* [DVD]. Silver Spring, MA, Discovery Communication Inc.

[7] Taipei Fire Department. 2000. *Memorandum of Rescuing Action at Dong-Hsing Condo in 1999 Taiwan Earthquake*., Taipei Municipal Government, Taiwan, R.O.C. (in Chinese).

[8] Lawrence, P. and J. Lorsch 1967. *Organizations and Environment*. Cambridge, MA, Harvard University Press.

[9] Thompson, J.D. 1967. *Organizations in Action*. NY, McGraw Hill.

[10] Tsai, J.S., C.C. Cheng and S.B. Chen. 2007 "Effective Coordination of Engineering Organization for Emergent Management," Final report, NSC E-96-2221-E006-226, National Science Council, Taiwan, R.O.C. (in Chinese).

[11] Jin, Y. and R.E. Levitt 1996. "The Virtual Design Team: A computational model of project organizations," *Comput. Math. Organ. Theor.*, 2(3):171-195.

[12] Galbraith, J. 1977. *Organizational Design*. Reading, MA, Addison-Wesley.

[13] Burns, T. and G. Stalker 1961. *The Management of Innovation*. London, Tavistock Press.

[14] NSF-IRIS 1989. "A report by the NSF-IRIS Review Panel for Research on Coordination Theory and Technology", Available from NSF Forms & Publications Unit.

[15] Malone T.W. and K. Croswton 1991. "Toward an Interdisciplinary Theory of Coordination, " Technical Report #20, Center for Coordination Science, Massachusetts Institution of Technology, Cambridge, MA.

[16] Malone, T.W. 1988 "What is Coordination Theory," Working Paper #2051-88, MIT Sloan School of Management, Massachusetts Institution of Technology, Cambridge, MA.

[17] Tushman, M.L. and D.A. Nadler 1978. "Information Processing as an Integrating Concept in Organizational Design," *Acad. Manage. Rev.*, 3(3): 613-624.

[18] Galbraith, J. 1973. *Designing Complex Organizations*. Reading, MA, Addison-Wesley.
[19] Campbell J.P. 1977. “On the Nature of Organizational Effectiveness,” in *New Perspectives On organizational Effectiveness*, P.S. Goodman and J.M. Pennings eds., San Francisco, CA, Jossey-Bass, pp. 13-55.
[20] Barnard, C.I. 1938. *The Functions of the Executive*. Cambridge, MA, Harvard University Press.
[21] Scott, R.S. 1977. “Effectiveness of Organizational Effectiveness Studies,” in *New Perspectives On organizational Effectiveness*, P.S. Goodman and J.M. Pennings eds., Jossey-Bass, San Francisco, CA, pp. 63-95.
[22] Scott, R.S. 2003. *Organizations: Rational, Natural, and Open Systems*. Upper Saddle River, NJ, Prentice Hall.
[23] Dooley, K. 2002. “Simulation Research Method,” in *Companion to Organizations*, J. Baum eds., Blackwell, London, pp. 829-848.
[24] Horri, T., Y. Jin and R.E. Levitt 2005. "Modeling and Analyzing Cultural Influences on Project Team Performance," *Comput. Math. Organ. Theor.*, 10(4):305-321.
[25] Carley, K. 1995. "Computational and Mathematical Organization Theory: perspective and directions," *Comput. Math. Organ. Theor.*, 1(1):39-56.
[26] Weber, M. 1924. *The Theory of Social and Economic Organization*. Free Press.
[27] March, J.G. and H.A. Simon 1958, *Organizations*, NY, Wiley and Sons Inc.
[28] ePM 2006. *SimVision Tutorial in electric version.* Austin, TX, e Project Management LLC.

A Regional Water Supplier's Seismic Response Plan as Part of the Organization's Overall Emergency Management Plan

Wally M. Lieu, Richard T. Shpall, Winston Chai

ABSTRACT

A water agency has the responsibility to ensure reliable delivery of high quality water to its customers, even in a seismically active region. The Metropolitan Water District of Southern California serves a population of 18 million people with a regional economy of over $800 billion. Water agency earthquake preparedness involves several components including inspecting and assessing existing infrastructure, retrofitting and strengthening to meeting current code requirements, and response procedures in the event of a moderate to major earthquake. Planning also extends beyond the agency's region to ensure an available water supply. This paper describes one agency's Seismic Response Plan under its umbrella Emergency Management Plan.

INTRODUCTION

The Metropolitan Water District of Southern California (MWD) was created by the legislature of the State of California in 1928 for the purpose of constructing and operating an aqueduct and ancillary facilities to transport water from the Colorado River to Southern California. Since that time MWD's facilities have been expanded to include a conveyance and distribution system stretching over 800 miles, 5 regional water treatment plants, and 18 reservoirs. MWD's facilities are functionally linked to those of the California Department of Water Resources which supplies MWD with a significant portion of its untreated water and are linked to the individual water retailers' systems which deliver water treated in MWD facilities to the end users. Much of MWD's approach to planning for seismic events is applicable to other agencies.

Water supplied by MWD serves 18 million people located in an area encompassing 5,200 square miles in seismically active Southern California. The area is crisscrossed by numerous faults of varying degrees of activity. This includes faults that have not ruptured in hundreds of thousands of years and the infamous San Andreas Fault which stretches nearly the length of California and whose estimated rupture recurrence interval in the Southern California segment is 140 years.

Wally M. Lieu, PE, Engineering Section Manager, Metropolitan Water District of Southern California, P.O. Box 54153, Los Angeles, California, 90054-0153

Richard T. Shpall, PE, Principal Engineer, Metropolitan Water District of Southern California, P.O. Box 54153, Los Angeles, California, 90054-0153

Winston Chai, Ph.D., SE, Engineer, Metropolitan Water District of Southern California, P.O. Box 54153, Los Angeles, California, 90054-0153

Southern California has experienced seven strong earthquakes (M6 to M6.9) and two major earthquakes (M7.0 to M7.9) in the last 40 years. The major earthquakes were centered in desert regions and caused only minor damage. Two of the strong earthquakes (San Fernando Earthquake, M6.7 in 1971 and Northridge Earthquake, M6.8 in 1994) caused damage to infrastructure and disrupted many lifelines including water service in selected areas. Water service was restored in few days to several weeks depending on the level of damage.

MWD's system was constructed over the course of the last 80 years in conformance with building codes in effect at the time. Over this period the codes have evolved as the industry's understanding of earthquake hazards has increased. The San Fernando Earthquake demonstrated the vulnerability of the water delivery system when one of MWD's treatment plants was severely damaged. A task force was formed after the earthquake to develop long-term strategies to systematically reduce MWD's seismic risk. A Seismic Program was established with the goal of maintaining water system operation immediately after a strong earthquake. To achieve the goal, a multifaceted approach was taken, to assess overall seismic vulnerability of MWD's systems, to upgrade structures to current seismic design standards and industry practice, and to formulate an emergency response plan to rapidly restore system operation in the event of system failures. This paper summarizes the activities undertaken by MWD to manage the risk associated with earthquakes.

SEISMIC PROGRAM

As an agency's system grows to expand service to the entire region, facilities are constructed to convey untreated water to users and to treatment plants, to build new treatment plants to meet the needs of the growing population, to maintain compliance with evolving water quality regulations, to store water to act as a buffer for short term changes in demand, and to build new pipelines to distribute treated water to an expanding service territory. The expansion of MWD's system is reflected in the construction of over 300 structures. Many of these structures were built prior to the development of modern building codes which address earthquake safety. The Seismic Program was undertaken to develop an organized approach to evaluate structures constructed to the older building codes and where necessary to begin a program of retrofitting structures.

The initial steps were to develop seismic design criteria for the re-evaluation that incorporate current code provisions and industry standards, and also address seismic issues unique to MWD. Existing facilities have been systematically evaluated and upgraded to maintain a comparable seismic performance level as that of a new facility. As the primary water supplier of the region, MWD has various types of structures to meet its operational needs including pumping, conveyance, storage, treatment and distribution. Although the ultimate goal of reducing seismic risk is the same, evaluation and design of different structures require different procedures to address specific issues. It is also important to minimize seismic risk of non-structural components such as equipment and piping in order to achieve the goal of maintaining system operation after earthquakes.

A seismic risk reduction program requires long-term planning and a large capital investment. It is crucial for the engineering staff to identify seismic deficiencies and quantify the associated risks so that upgrading and strengthening can proceed timely and effectively.

Development of Seismic Design Criteria

Once the seismic vulnerability is identified, an organization needs to determine an acceptable level of seismic risk and establish a seismic performance objective for its facilities to better achieve its mission and fulfill its responsibility to the general public. A seismic performance objective consists of two components – seismic hazard level (demand) and structure's seismic performance level (capacity). A critical task of the Seismic Program is to develop design criteria that provide procedures to quantify the seismic hazard level and specify the expected seismic performance level.

MWD's seismic design criteria are based on the current California Building Code (CBC) [1], with necessary modifications to achieve the goal of its Seismic Program. For example, MWD's facilities related to water delivery are intended to be designed for a seismic performance level higher than that required by the CBC [1] because it would supply the majority of the water used for post-earthquake disaster relief and fire suppression of the entire region. The building code provides minimum requirements for design and construction of new buildings but in many ways, is not adequate for evaluation and retrofit design of existing structures constructed in different eras with different seismic deficiencies. It is a unique challenge to a regional agency to develop the seismic design criteria that cover the entire inventory of structures including administrative buildings, pump stations, aqueducts, water bridges, reservoirs, inlet/outlet towers, treatment plants, power plants, warehouses, and maintenance shops. The seismic design criteria need to incorporate pertinent national standards and guidelines from various government agencies and research institutes to supplement the seismic provisions in the local building code. In developing the criteria, the goal of the seismic program must be clear and precise but the procedures to achieve it could be flexible and should be constantly updated to reflect new findings and incorporate new technologies.

For new design, facilities are divided into three groups - those related to water delivery, those not related to water delivery but essential to MWD's business operation and those not in the first two groups. The first group consists of facilities that are necessary for pumping, conveyance, storage, treatment and distribution of deliverable water. Facilities with functions to support the operation of this group, such as access, power and communication are to be designed with the same criteria. The second group consists of mainly office buildings and supporting facilities for business operation. In general, loading, application, and design methods conform to the CBC [1] which refers to ASCE 7 [2] for detailed requirements. Facilities in the three groups are classified as essential, important, and regular and correspond to Occupancy Categories IV, III, and II respectively as specified in ASCE 7 [2]. A brief explanation of theses classifications is that an essential facility is intended for immediate occupancy and a regular facility is intended for life safety, respectively, under the design earthquake. The seismic performance of an important facility is expected to be in between. The design earthquake is two-thirds of the Maximum Considered Earthquake (MCE) as defined in ASCE 7 [2].

The seismic design criteria for retrofit of existing facilities are based on CBC [1] and ASCE 41 [3]. The seismic hazard level is essentially the same as that for new facility design with the design earthquake equal to two-thirds of MCE. Although some guidelines and standards allow a reduction in seismic design forces for existing building evaluation with the assumption that the remaining life of an existing building is less than that of a new building, each agency should determine whether to allow such reduction since many existing facilities may need to be continuously operated beyond their original design life. Unlike the criteria for new facility design, only two seismic performance levels are specified for existing facility retrofit.

This is mainly because the current retrofit design documents do not provide clear criteria for performance levels between immediate occupancy and life safety. Although the facilities listed under Occupancy Category III are important to resume MWD's business operation after earthquakes, they are less critical to its core mission of continuous water delivery. Therefore, existing facilities related to water delivery are designed to meet the immediate occupancy performance level and the rest of the existing facilities to meet the life safety performance level.

Other non-building structures such as basins, tanks and inlet/outlet towers have seismic design criteria based on applicable provisions in CBC [1] and industry standard practice. Despite varied design procedures, MWD's seismic design criteria ensure a uniform seismic hazard level for all these structures. Design procedures specified in ACI 350 [4], in API 650 [5] and AWWA D100 [6], and in various publications [7][8][9] by US Army Corp of Engineers provide procedures and guidelines for design of concrete water containing structures, steel tanks, and inlet/outlet towers, respectively.

Evaluation of Existing Facilities

Many of MWD's facilities were built in the early and mid 20th century and do not meet minimum seismic design requirements specified in modern building codes. Without upgrading these seismically vulnerable facilities, MWD risks losing its capability to deliver water for a substantial period of time after a strong earthquake. MWD is committed to reducing the seismic risk but realizes it must utilize its limited resources and finances efficiently. Criteria were established to help prioritize the retrofit effort and procedures were developed to systematically evaluate the existing facilities and determine necessary actions. Seismic evaluation of an existing MWD structure consists of three phases: rapid evaluation, detailed evaluation and final retrofit design. If the rapid or the detailed evaluation demonstrates that a structure is adequate for resisting the design earthquake, the subsequent evaluation/design will not be necessary.

Prioritization

Much of the basic seismic design details and requirements that are considered to be necessary today were not required in earlier versions of the building code. It is generally recognized that structures designed and constructed per codes prior to the 1973 Uniform Building Code (UBC) would have little design features for earthquake resistance. Lessons learned from previous earthquakes, especially the 1971 San Fernando Earthquake, prompted code changes on seismic design provisions with requirements for ductile detailing and consideration of dynamic characteristics of structures. After the 1989 Loma Prieta Earthquake, more modern seismic design requirements were incorporated into the building code to reflect the findings and lessons learned in the 1980's. The consensus of the industry is that the structures built in 1990's and later which should be designed and constructed in accordance with the 1988 or later versions of UBC provide reasonable assurance of withstanding a code level earthquake without catastrophic structural failure. Considering that MWD has a vast inventory of existing structures to be evaluated, the decision was made to prioritize structures built before early 1990's. Facilities related to water delivery were given priority over other structures. Criteria used to prioritize water delivery facilities include importance to the system (amount of water handled by a facility), lack of redundancy (inability to replace the lost capacity), a facility's vulnerability to earthquake damage and potential loss of life.

Rapid Evaluation

The initial assessment of existing facilities involves a rapid evaluation with the goal of quickly determining whether a structure is seismically deficient. Once a seismic deficiency is identified, the structure is categorized as seismically deficient and the rapid evaluation is complete. Simpler procedures such as drawing review, visual inspection and quick hand calculations are typically applied to the rapid evaluation. The American Society of Civil Engineers Seismic Evaluation of Existing Buildings, ASCE 31 [10] provides procedures to evaluate existing facilities. The Tier I evaluation specified in ASCE 31 provides engineers a simple procedure to identify seismic deficiencies by answering a series of questionnaires and is appropriate for this phase. Since the goal of this phase is to quickly screen the entire inventory of existing facilities for potential seismic deficiencies and the cost is relatively small, the expenditure is usually covered under operation and maintenance (O&M) budget to avoid lengthy approval process for a capital project. More than 97% of MWD's existing facilities have gone through the rapid evaluation process.

Detailed Evaluation

In the detailed evaluation phase, a structure identified with at least one seismic deficiency by the rapid evaluation is thoroughly evaluated to identify all the seismic deficiencies. Feasible retrofit schemes are developed during this phase to mitigate the identified deficiencies. A report documenting evaluation procedures and summarizing findings and recommendations is prepared at the completion of the evaluation. More advanced procedures such as finite element modeling and comprehensive structural calculations are applied in this phase. The Tiers II and III evaluations specified in ASCE 31 [10] provide applicable methodologies appropriate for this phase. Since a detailed evaluation would take longer and cost more to complete, it is usually budgeted as a capital project and would require approval from upper management to proceed.

Final Retrofit Design

One of the feasible retrofit schemes developed in the detailed evaluation would be selected for the final retrofit design. Construction documents including design drawings and specifications are prepared for retrofit construction. ASCE 41 [3] provides analytical and design procedures for seismic retrofit design of existing structures. Other well-established procedures based on national recognized methods of analysis in accordance with principles of mechanics may be applied to the retrofit design of existing facilities. Final design and construction of a seismic retrofit project would require substantial investment and must be incorporated into the overall financial planning of the agency. MWD's existing facilities that went through the detailed evaluation process are in various stages of retrofit work from planning for final design, under final design, planning for construction, under construction to completed construction. A completed project is shown in Figure 1 which demonstrates one of MWD's pumping plant during and after its retrofit construction.

Inspection of Non-Structural Components

Improving seismic safety of non-structural components is a critical step toward achieving the goal of maintaining water system operation after earthquakes. In particular, to ensure important equipment is adequately anchored and critical pipes/conduits/ducts are properly braced and supported. Since modifications are constantly made at facilities, it is important to conduct periodical inspections on existing equipment and pipelines to ensure that anchorage and bracing remain adequate. Regulations require equipment and pipes that have a potential of off-site release of hazardous materials during a strong earthquake be inspected periodically (e.g. every five years). An agency may conduct its own inspection of non-structural components in concurrence with the mandated inspection to more efficiently utilize its resources.

Figure 1. Pumping Plant During and After Retrofit Construction

System Vulnerability Assessment

As discussed above, MWD is continuing the evaluation of older structures against current design practices. Despite this proactive program, seismic risks remain: actual forces associated with an earthquake may exceed the design levels, and pipeline and conveyances are subject to damage caused by ground displacements, liquefaction etc. In addition, the State water system, upon which MWD is dependent is subject to the same hazards and may be disabled causing an interruption in the supply to MWD.

To assess the risk associated with these factors, a system vulnerability assessment was conducted to estimate the damage that could result from earthquakes in Southern California and the corresponding potential impacts on MWD water deliveries. Evaluation scenarios were developed assuming an earthquake on each of the major Southern California faults and considered fault ruptures of varying levels of intensity. Then considering the proximity of MWD's facilities to each earthquake fault, estimates were made of the portions of the system potentially impacted. Based on the affected system components, the corresponding influence on system operation was assessed. The analysis includes potential impacts of earthquakes on components of the State system which are critical to the performance of MWD's system.

This study was effective in characterizing the potential magnitude of damage that could result from earthquakes within MWD's service area and the corresponding potential impacts on MWD water deliveries. Through the analysis a number of projects and procedures were identified that could mitigate earthquakes. The results of the vulnerability assessment can also be useful to agencies dependent on MWD's system in determining the need for localized emergency storage and interconnections to other local agencies.

EMERGENCY RESPONSE PLAN

An Emergency Response Plan (ERP) is a staffing and assessment plan designed to establish an organized and systematic response to earthquakes or other emergencies caused by natural disasters or other unavoidable circumstances. MWD's ERP complies with the Standardized Emergency Management System (SEMS) and the Incident Command System (ICS) and has adopted the National Incident Management System (NIMS). It establishes an Emergency Response Organization (ERO), an operational hierarchy intended to expedite recovery after an emergency by establishing communication channels and logistics responsibilities. The ERP creates a general framework to quickly evaluate the emergency situation and coordinate a response.

The ERP breaks down into three different areas of emergency response which are built upon a foundation of industry recognized practices and standards: Crisis Management and Emergency Response (CM/ER), Business Continuity (BC), and Information Technology (IT) Disaster Recovery (ITDR). The staff from these three areas of emergency response is responsible for returning MWD to normal operating conditions. The three areas of emergency response are represented by the ERO and their responsibilities in the event of an emergency.

Emergency Response Organization (ERO)

Dependent on the magnitude, earthquakes have the potential to affect a large area from a single event. Therefore, the ERO's framework is designed to cover the three areas of emergency response and minimize MWD's vast service area through effective communication and coordination. The primary assessment is done through eight geographically based Incident Command Centers (ICCs) and three function based ICCs. In addition, the Information Technology group has a designated IT Incident Command Post (IT-ICP). These eleven ICCs and the IT-ICP coordinate with the Emergency Operations Center (EOC) to develop a response strategy (Figure 2).

CM/ER is designed to quickly assess the impact of an earthquake on facilities and to restore an organization's ability to deliver water to member agencies. MWD's CM/ER is represented through the eight geographically based ICCs responsible for organizing personnel and distributing resources in order to restore MWDs operations. Five of the ICCs are responsible for MWD's five treatment plants while the other three are divided into regions that are responsible for assessing conveyance facilities. These eight ICCs direct the Field Command Posts (FCP) in their specific regions ensuring that the proper resources are available where needed.

The remaining three ICCs and the IT-ICP cover the BC and ITDR response and ensure that MWD continues to function as a resource provider. BC is represented by the Business ICC (BICC), the Water Quality ICC (WQICC), and the Headquarters ICC (HICC) and is designed to ensure continuation of critical business processes. The Business ICC analyzes impacts to MWDs business functions caused by failure of major business systems or power outages, assesses the risks to the organization, and ensures continuation of critical business functions under emergency situations. The Water Quality ICC is responsible for analyzing the impacts to MWD's water quality functions and to ensure that any water quality concerns are reported to the EOC. The Headquarters ICC is responsible for restoring the functionality of the headquarters administrative facility. The IT-ICP is responsible for the restoration of IT infrastructure.

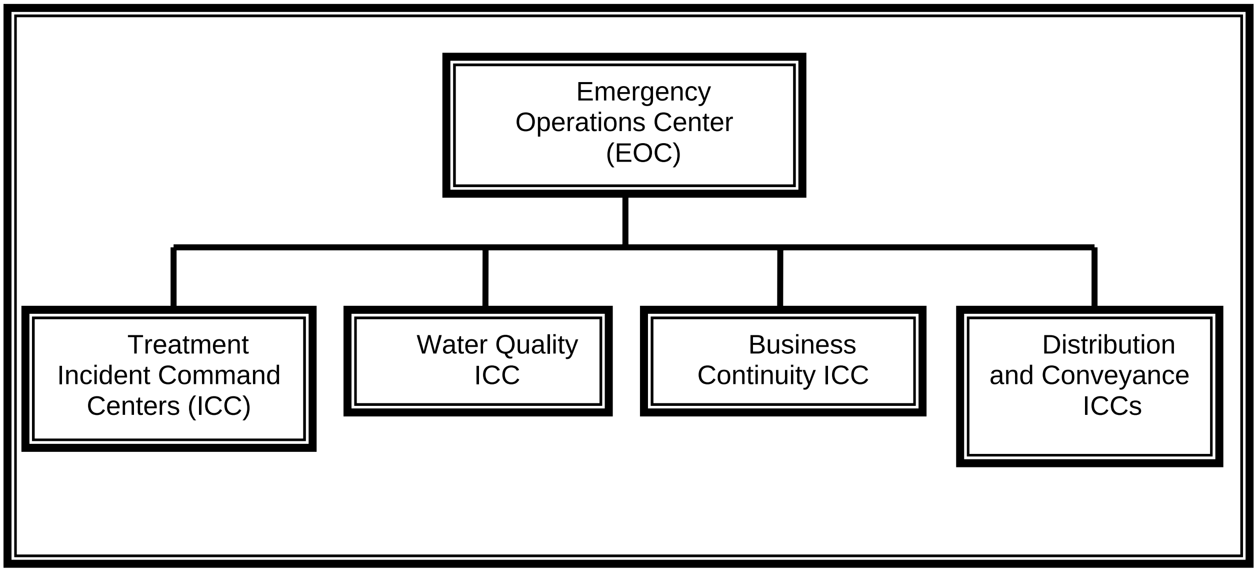


Figure 2: Emergency Response Organization

The EOC organizes and prioritizes the emergency response for MWD as a whole. The EOC will manage coordination between the ICCs to provide support for individual ICCs requiring resources beyond their capabilities. The EOC also coordinates with outside agencies including local, regional, and state emergency management groups. Lastly, the EOC is responsible for distributing critical information to the media and other relevant organizations as well as coordinating mutual aid and mutual assistance programs with other agencies.

The ERP and its effectiveness in response to an earthquake is dependent on the readiness of those within the ERO. Therefore, MWD organizes annual drills in compliance with Federal Emergency Management Agency and California Exercise Guidelines. Groups within the ERO are required to conduct a tabletop (discussion based) exercise or a functional exercise every two months with at least one functional exercise every 12 months. The functional exercise can be simulated or full-scale involving multiple agencies or jurisdictions.

Communication

Lessons learned from the 2005 hurricane season and the magnitude of devastation to the Gulf Coast region demonstrated the necessity for diversified forms of emergency communication capabilities. In the event of an earthquake, it should be anticipated that some communications networks could be unavailable or unreliable due to heavy traffic and/or infrastructure problems. Therefore, additional radio networks and communication capabilities should be established to ensure communication during an emergency and strengthen emergency response and recovery programs.

MWD has an extensive phone and network radio system that operate very efficiently under normal conditions. MWD's 2-way radio system can communicate between facilities and from facility-to-field unit via two internal radio networks on VHF and UHF frequencies.

Additionally, MWD has established a primary emergency amateur radio communications channel and provides amateur radios in many of the ICCs and the EOC. Lastly, MWD has also invested in a satellite phone system because they have shown to deliver a reliable form of communication when all other means are lost.

Logistics

To communicate and guide MWD's response to an earthquake, the EOC and the ICCs would develop an Action Plan. The objectives of the Action Plan are to protect against threats to life and property and contain the progress of worsening incidents. The Action Plan is developed during an initial operational period, generally a few hours, in which a preliminary damage assessment report is developed. This allows the EOC and the ICCs to determine what damage is most critical and where to provide the necessary resources.

Plan Activation and Response

The ERO contains automatic activation standards for earthquakes as well as a list of responders. Activation of the ERO is dependent on the location of the earthquake as well as the magnitude as shown in Figure 3. If there is an earthquake of magnitude 5.5 or higher within MWD's service area or along the Colorado River Aqueduct (CRA), 6.0 or higher within 30 miles of the service area or the CRA, or 7.0 or higher south of the cities of Baker and Bakersfield, a full activation of the ERO would occur (Figure 3). Based on set protocols as shown in Figure 4, emergency personnel would report to their designated areas and activate the EOC, the ICCs, the BICC and the IT-ICP.

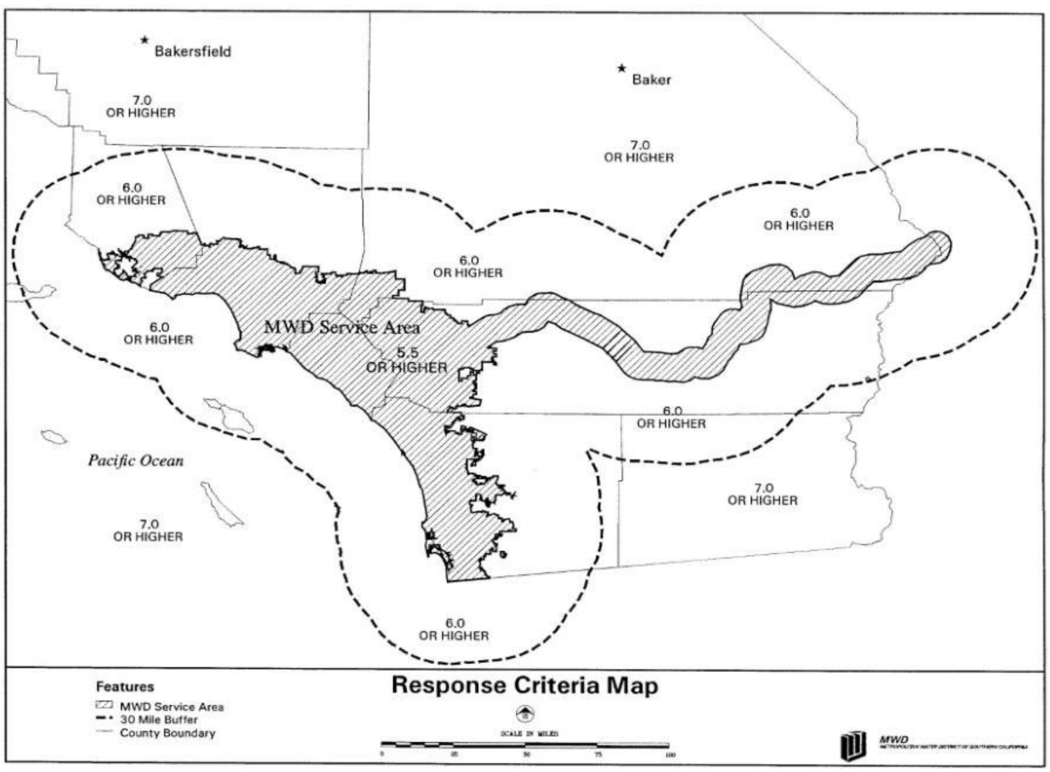


Figure 3: Standards for Automatic Activation of Emergency Response Plan

Once activation occurs, pre-assigned primary and back-up patrollers would begin assessment of conveyance and distribution facilities. The patrollers would assess damage to their pre-defined routes providing an overview of the entire water system within a few hours of the event. A first run of the route would give a quick assessment looking for obvious damage or leaks; patrollers would not stop to make repairs unless there is an immediate risk of injury. A second run would cover the route in more detail reading meters and piezometers.

Once the initial assessment is reported to the ICCs, Damage Assessment Teams (DAT) would be mobilized to prioritized locations as determined through the Action Plan. The DATs cover multiple engineering disciplines with at least two personnel for each discipline. The DATs would investigate the damage and determine if the response requires a short-term workaround or a long-term repair.

Earthquake Recovery Operations

MWD has invested heavily in the hardening of critical system components against earthquakes. Nonetheless, contingencies must be made in the eventuality that for any given earthquake scenario that some failures might occur. MWD has developed a substantial in-house capability to implement repairs.

This capability was primarily developed to meet MWD's needs to perform routine system maintenance and to repair random failures in the system. This capability is currently being upgraded to enable MWD staff to address repairs anticipated from a strong earthquake and to initiate repairs anticipated from a major earthquake. The activities undertaken to implement repairs are managed and coordinated under the ERP.

MWD staff is skilled in the use of the construction equipment. Depending on the level of effort, the staff may be augmented by temporary construction crews. MWD frequently is undertaking large construction projects at several MWD sites. By contractual arrangement MWD has the authority to redirect the contractor's forces to assist in conducting system repairs following an earthquake. In addition, MWD is a participant in several mutual aid programs that allow MWD and its member agencies to assist each other with staff, materials and equipment. The California Water/Wastewater Agency Response Network (Cal-WARN) program is a mutual aid program that operates on a State-wide level providing a mechanism for utilities throughout the State to share emergency resources during and following disasters.

CONCLUSION

Preparation for earthquake hazards requires actions by agencies on multiple levels. It starts from a long-term strategy to mitigate seismic risk which consists of assessment of seismic vulnerability, development of seismic design criteria, evaluation of existing facilities, implementation of retrofit designs and periodic inspections. Recovery from the effects of an earthquake depends on a well designed emergency response plan and access to a skilled workforce and construction equipment and materials. Periodic exercises ensure the readiness of the team to respond to earthquakes or other emergencies.

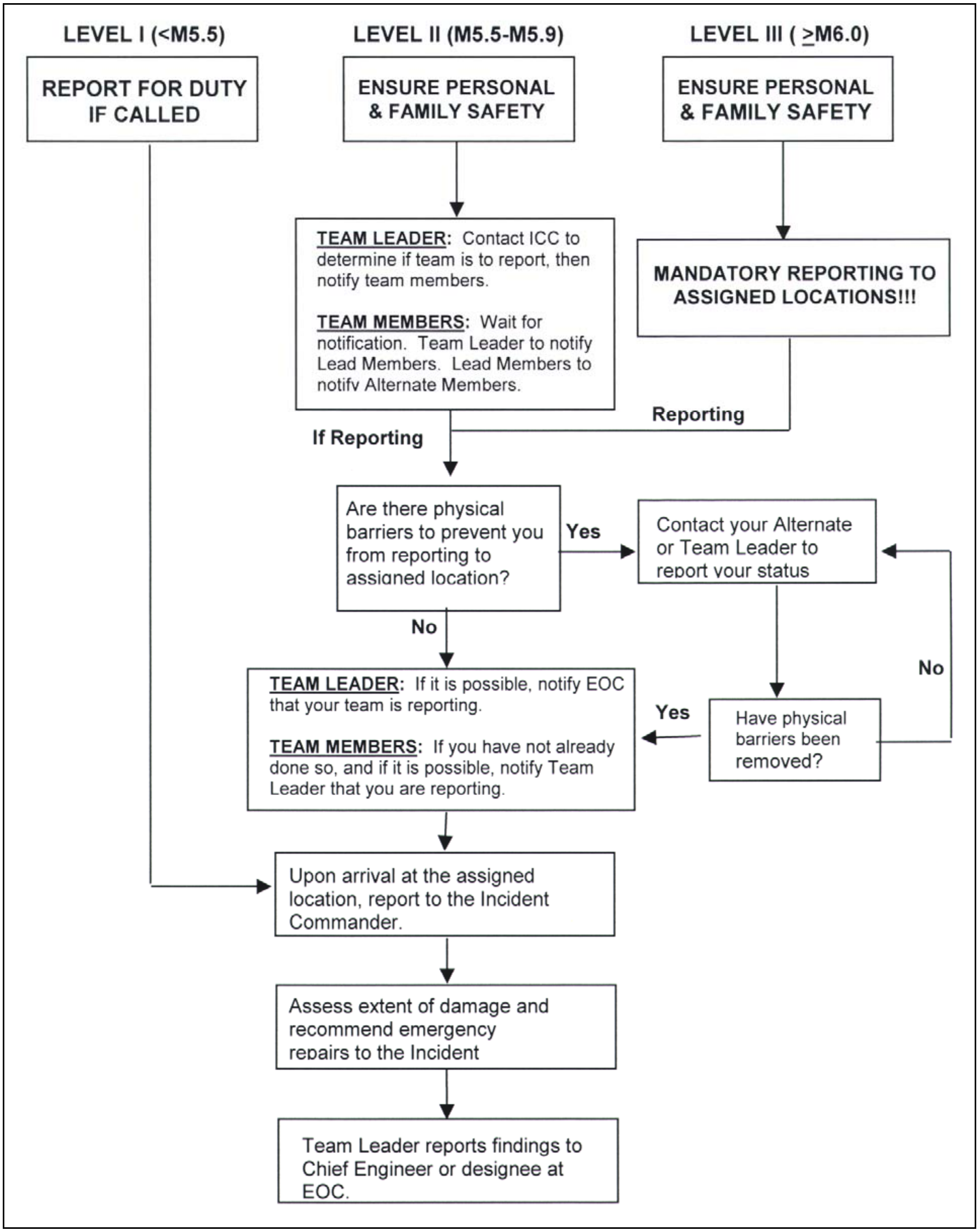


Figure 4: Earthquake Emergency Response Protocol

REFERENCES

1. California Building Standards Commission. 2007. “2007 California Building Code.”
2. American Society of Civil Engineers. 2005. “Minimum Design Loads for Buildings and Other Structures.”
3. American Society of Civil Engineers. 2006. “Seismic Rehabilitation of Existing Buildings.”
4. American Concrete Institute. 2006. “Code Requirements for Environmental Engineering Concrete Structures and Commentary.”
5. American Petroleum Institute. 2008. “Welded Tanks for Oil Storage, 11^{th} Edition.”
6. American Water Works Association. 2005. “Welded Carbon Steel Tanks for Water Storage.”
7. US Army Corps of Engineers. 1999. “Response Spectra and Seismic Analysis for Concrete Hydraulic Structures.”
8. US Army Corps of Engineers. 2007. “Earthquake Design and Evaluation of Concrete Hydraulic Structures.”
9. US Army Corps of Engineers. 2003. “Structural Design and Evaluation of Outlet Works.”
10. American Society of Civil Engineers. 2003. “Seismic Evaluation of Existing Buildings.”

Introduction of Emergency Water Supply Facilities in Taipei Metropolitan

H.H. Ju[1] K.P.Chang[1] S.J. Shue[2] S.L Teng[2] S.C. Lin[2]

ABSTRACT

Sizable disaster could damage water supply facility and transmission and distribution pipeline and result in disruption of water supply in short period of time, and the interests of the public in maintaining life supporting water supply would be jeopardized. In order to be ready for such incidents, Taipei Water Department (TWD) built emergency water supply facilities gradually since 2004 and up to 2008, 30 sets of such facilities have been completed. It is estimated that these facilities are capable for supplying 280,000 cubic meter drinking water to 3.87 million people in TWD service area for 24 days. Subsequently, in coupling with the disaster preventing park plan of Taipei City Government, 3 new water storage tanks of pipe shape and 4 reinforced concrete storage tanks will be built in the following years. It is conceived that shut-off valve is to be installed to the connecting pipeline of storage tank which can be shut off automatically at occurrence of disaster and retain water in storage tank. Thus, people can take water from the storage facilities for life supporting purpose.

In this paper, the author will recount the target, content and present situation of TWD Emergency Water Supply Plan. We also present our life-supporting storage tank designs, emergency shutoff valves, flexible pipes , model test and the problems we encountered. Finally, the prospect will be discussed as well.

Key Words: Emergency Water Supply Plan, Emergency Water Storage Tank

1. INTRODUCTION

Taiwan is located on the earthquake belt of Pacific Rim and earthquake is a common thing. When a strong earthquake occurs, it could damage to water supply equipment and pipeline and interrupt normal water supply for a short period of time. It could cause growth of pathogens and initiate epidemics. In view of the above, in the city disaster prevention and protection facilities, the establishment of emergency water storage facilities is one of the most important things.

In 2004, Taipei City Government instructed Taipei Water Department (TWD) to review the places that are capable to store water and to plan for necessary facilities to ensure water supply services will be maintain despite of emergency situation. Following this instruction, TWD drew an Emergency Water Supply Plan to establish emergency water supply stations beside distribution basin, to draw water from water distribution mains, and to build emergency water storage tank in disaster preventing park of Taipei City.

[1] Sub-section Chief, Taipei Water Department, Taipei City Government, 3F, No.92, Sec. 4, Roosevelt Rd., Taipei, 100, Taiwan (R.O.C.), E-mail: hann@twd.gov.tw

[2] Engineer, Taipei Water Department, Taipei City Government, 3F, No.92, Sec. 4, Roosevelt Rd., Taipei, 100, Taiwan (R.O.C.)

2. EMERGENCY WATER SUPPLY PLAN

2.1. Target of the Plan

The target of this plan is to establish emergency water supply storage facilities, which enable TWD to maintain basic drinking water supply to the public at 3 liter per capita for 4 weeks, when a sizable disaster damages the water supply system.

2.2. Content of the Plan and Situation of Execution

The emergency water supply drawing facilities to be established under this plan may be divided into 3 types. The first one is to establish emergency water supply drawing station beside the distribution tank. The second one is installing water drawing equipment which could draw clear water from the large-size pipes through the air relieve valves. The third type is establish underground pipes or reinforced concrete water tanks in the shelter parks. The following are the brief description of the 3 type emergency water supply facilities.

2.2.1. Emergency Water Supply Drawing Station of Distribution Tank

The water stored in the distribution tank can first be pumped to storage tank or supply tower above ground and then flow to the drawing ports through branch pipes. Citizens can receive water from the faucets of drawing port directly. Also, in order to increase the flexibility of water supply, the drawing port for water lorry will be installed to large distribution tank to facilitate the supply to neighboring area or for emergency use. Fig. 2-1 is the illustrative drawing of drawing water. So far, TWD has completed Emergency Water Drawing Station in 11 distribution tanks.

2.2.2. Emergency Water Drawing Station of Distribution mains

Tap-water pipelines are almost buried under ground. The transmission and distribution pipeline constructed in open-cut method are mostly 1.2 M deep under the ground. But in cases of passing through road intersections, passing river with siphon method, or in tunnel boring method, the depth will be far more than 1.2 M, and will be relative lower than those pipes constructed in open-cut method. In outage incidence, the relative lower section will be deposited with substantial volume of water and installing water drawing facility to these sections, it will be a source of emergency water supply. The illustrative Drawing is given as Fig. 2-2 (Installed with emergency water drawing equipment and pump(s) will be placed through the input hole of distribution mains for drawing water direct to the citizen or to water lorry to support neighboring area.) TWD has presently completed 19 Emergency Water Drawing Stations of distribution mains as well.

2.2.3. Pipe Form Storage Tank and Concrete Storage Tank

(1) Pipe-Form Storage Tank

This is large diameter pipeline buried under park, greens, school or road / sidewalk .The length is from few meters to more than 10 meters and in diameter of 1500 mm to 2400 mm. If the length is 10 meters, the capacity will be 18 – 45 cubic meter. The pipe-form storage tank is hermetical and connected with the distribution pipes, so that water can flow in and out of it at normal time. Emergency shut-off valve shall be installed to the connecting pipe, which can be used as manhole for maintenance. At occurrence of sizable earthquake, the shut-off valve will be closed automatically to keep the water inside the pipes. By using drawing equipment, the stored water can be drawn out as emergency water supply for the neighboring citizens. TWD plans to install 3 pipe-form emergency water storage tanks in Shelter Parks and they are under construction. Two are almost completed.

(2) Reinforced Concrete Emergency Storage Tank

The capacity of reinforced concrete emergency storage tank is large than Pipe-Form water storage tank and is generally more than 500 cubic meter. It is installed with connecting pipe to connect with the neighboring distribution pipe. It is an open-to –atmosphere facility and therefore, boosting facility is required for pumping deposited water from the tank into water supply system and to maintain the freshness of water deposited. Similarly, emergency shut-off valve is installed to the connecting pipe. In normal days, it is used as system distribution tank in supplying water to users. At occurrence of sizable earthquake, the emergency shut-off valve will be closed automatically and keep the water inside the storage tank. Water drawing equipment can be used to draw water out as emergency supply of water. TWD plans to build 4 reinforced concrete emergency storage tanks in 4 shelter parks.

3. PIPE-FORM EMERGENCY STORAGE TANK

3.1. Principle of design and key points

3.1.1. Principle of design

In open-cut method, place enclosed pipe-form storage tank under ground of park. The material of storage tank shall be ductile cast iron or steel and both shall meet quake resisting, water tight, pipe-tank water retention ability and fall-off resistance. Emergency shut off valve shall be installed to the connection pipe between storage tank and existing distribution pipeline. At occurrence of earthquake, the emergency shut-off valve will be closed automatically and retain water inside the emergency storage tank, while the regular water flow will be maintained as usual in the pipeline network. Water drawing facility can be used to draw water out for emergency use of neighboring residents.

At normal days, the water stored in the tank is a part of the distribution pipeline and is maintained fresh all the time. At top of storage tank, two drawing port is fitted to be drawing port for emergency supply as well as firefighting hydrant. Remote monitoring system will be installed to transmit back the data of conditions of valves and storage tank to TWD monitoring center and Taipei City Fire Department. A store room shall be set up in park for storing diesel pump, manual pumping facility, emergency water supply equipment (water supply rack and hose), portable ventilator, and portable gas detector, for assembly and application within the shortest possible time.

3.1.2. Design Steps

In designing pipe-form emergency storage tank, the following are main items to be included in consideration ：

(1) Capacity of storage tank
(2) Checking of Pipe-form storage tank section: thickness of pipe wall, internal and external pressure, stress and deformation rate
(3) Checking of pipe axial direction: fall-off resistance
(4) Checking of buoyancy and settling
(5) Quake resistance: shall be meeting the requirements of “Building Quake Resistance Design Specification and Detail Interpretation” with regards to quake resistance requirements
(6) Capacity of Air Release Valve
(7) Support and soil improvement method for construction
(8) Emergency Shut-Off Valve Set

3.1.3. Model Experiment

In order to ensure the safety of water quality inside the tank, keeping away pollution due to idled water, TWD made a 1/10 storage tank model (Fig. 3-1) to simulate the flowing of water to find out the velocity and water replacement status with change in time.

In this experiment, we used colorimetric method to observe idle of water. We store firstly pigment contained water in the model, then pure water is introduced into model to replace pigment contained water. The pigmented water in the storage tank model must be repelled before the replacing rate (inflow volume / storage tank capacity) exceeds 6.

After 6 tests of different velocity, the finding is –"when the velocity is fast, the replacing time is short; on the other hand, when the velocity is slow, the replacing time is longer. The results of experiment are shown in Fig. 3-2.

3.2. Construction Steps and Precautions

3.2.1. Construction Steps

Main construction items can be divided into the building of storage tank, connection pipe, control panel and power supply system. The inspection items of storage tank are:

(1) Outer appearance and dimensions
(2) Coating: Coating of water storage tank includes sand blasting and rust removing both inside and outside, 2 primer coats, 2 epoxy coats for inside, and 2 primer coats and anti-corrosion cladding for outside.
(3) Non-Destructive Testing: Welding channels of storage tank must be X-Ray inspected.
(4) Material test: Tolerable stress of material and radioactive-free inspection.
(5) Water Quality Safety: Sampling at site and shall meet with the drinking water standard of TWD.
(6) Pressure and Leak Test: Pressure test must be tested to 7.5 kg/cm2 and last for 30 minutes and free of leak to be qualified. Leak test shall be tested to the pressure of 5kg / cm2 for one hour and is qualified if the leak is less than the calculated value.

3.2.2. Construction Cases

TWD has presently 2 pipe-form storage tanks under construction. Take Jing-hua Park project as an example （Fig. 3-3）. The main specifications are described as the following:

(1) Capacity: 100 M3
(2) Internal water pressure: 7.5kg/cm2
(3) Type of storage tank: Underground, total enclosed tank
(4) Material of storage tank: Rolled Steel (SS400)
(5) Circulating equipment: inflow and outflow pipes, emergency shut-off valve, air release valve
(6) Power Supply system: normal civilian power system
(7) Signal transmission: Installed to operate with valves
(8) Water drawing manner: Diesel Self-Primed Pump and Manual Pumping Facilities
(9) Others: Portable ventilator, portable gas detector, lighting and emergency power supply, store room and name plate (incl. brief illustration of the project and functional description).

3.2.3. Precautions

(1) Erection and lifting should be executed by heavy equipment, so the traffic conditions by the construction site must be fully aware.
(2) The operation space must be investigated in advance to assure the length and burying depth of storage tank is proper.
(3) Prior to construction, deep cutting must be conducted to find out the conditions of other possible pipelines barriers to assure with sufficient space.
(4) When constructing above soft soil, firm and strong foundation must be made to avoid any soil sinking.
(5) Since the construction sites are mostly highly populated, retaining support must be made properly .Noise and the cracking of nearby buildings shall be watched too.

4. PROSPECT

The emergency water supply plan of TWD is planned basis the existing water supply system, and the emergency water supply stations are located based on distribution tanks and available distribution mains or in shelter parks, they could not be located evenly and the density will not be sufficient. In future, to fulfill the emergency water supply stations can be placed evenly , TWD shall be attended by finding more feasible places to build those stations, so that the system may be as more perfect as possible.

REFERENCE:

[1]. Report for Leak Improvement and Pipeline Replacement Plan, and Emergency Water Supply System for Earthquake Resistant in Tokyo(2004) , Taipei Water Department (2007) Taipei, ROC

[2]. Study on Establishing Emergency Water Supply Facilities to the Existing Water Supply System, Taipei Water Department (2007) Taipei, ROC

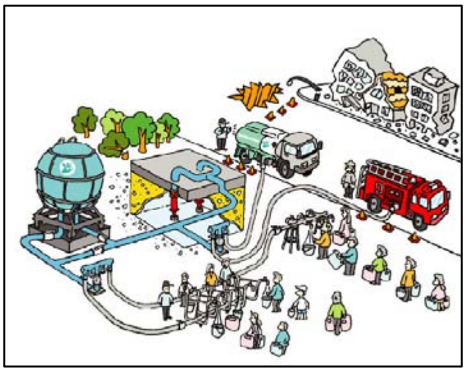


Fig. 2-1 Emergency Water Drawing from Distribution Pond

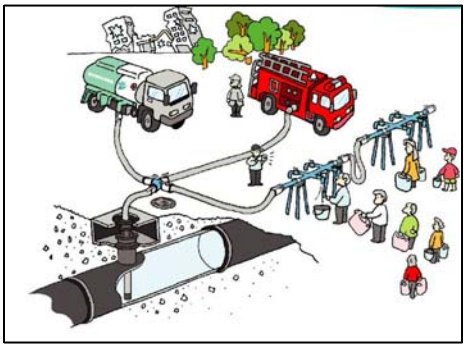


Fig. 2-2 Trunk Line Emergency Water Drawing Station

Fig. 3-1 Pipe-Form Emergency Storage Tank

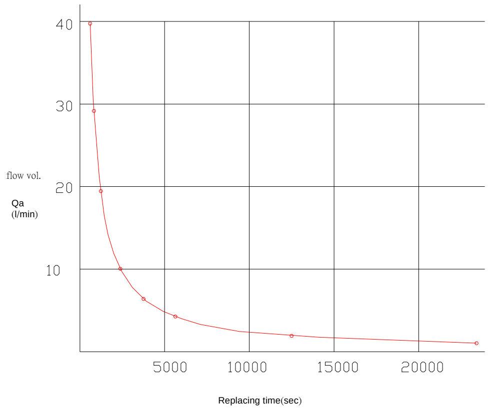


Fig. 3-2 Results of Experiment

Fig. 3-3 Site Construction

Fig. 3-3 Site Construction

Practical Reinforcement Action of Risk and Crisis Management with Participatory Planning Using a Workshop Method - Emergency Response to Natural Disasters

Masao Kadowaki, Nagahisa Hirayama, Daiji Nagashio, Masakazu Mihara

ABSTRACT

Hanshin Water Supply Authority prepared the disaster planning manual and improved seismic capacity of facilities to cope with each crisis phenomenon. However, for the artificial menace by terrorism and natural disasters by climate change, the Authority thought about not coping only by the exiting manual and improvement of facilities. Now, the Risk and Crisis Management Plan was developed through a planning process using a workshop method with participation of the Authority's staff.

The Risk and Crisis Management Plan has a hierarchy of "strategic plan", which sets a goal and develops objectives, policies/measures, projects/actions, to accomplish this goal. In a workshop, firstly, the Authority decided the goal, "To supply secure and reliable water in any crisis or emergency". Secondly, objectives of the risk and crisis management plan were set for each of five categories, which were 1) organization and system, 2) human resources, 3) information management, 4) water supply facilities and logistics, 5) recovery and reconstruction.

This paper describes the measures and actions to deal with natural disasters including earthquakes that are stated in the Risk and Crisis Management Plan, and its relationship with the current facilities improvement plan. It also mentions the advantages of adopting participatory planning using a workshop method to develop the Risk and Crisis Management Plan.

Masao Kadowaki, Waterworks Engineer, Construction division, Hanshin Water Supply Authority, 3-20-1 Nishiokamoto, Higashinada, Kobe, Japan 658-0073

Nagahisa Hirayama, Associate Professor, Kyoto University, Graduate School of Engineering, C1-3-182 Kyoto-daigaku-katsura, Nishikyo-ku,, Kyoto, Japan 615-8540.

Daiji Nagashio, Waterworks Engineer, Construction division, Hanshin Water Supply Authority, 3-20-1 Nishiokamoto, Higashinada, Kobe, Japan 658-0073

Masakazu Mihara, Waterworks Engineer, Construction division, Hanshin Water Supply Authority, 3-20-1 Nishiokamoto, Higashinada, Kobe, Japan 658-0073

INTRODUCTION

The Hanshin Water Supply Authority is a municipal utility that supplies drinking water to 2.5 million people living in the 719 km^2 Hanshin Area, southern Hyogo Prefecture. Since it started supplying water in 1942, the Authority has expanded its facilities piecemeal. It is now responsible for 187 km of pipelines and has a total facility capacity of 1,128,000 m^3/d. The facilities are roughly classified into two systems fed by the Yodogawa and Daido Intake Pump Stations. When the ongoing construction of Amagasaki Water Treatment Plant (WTP), which is currently operating at half its designed capacity, is completed, the Authority will have a total facility capacity of 1,289,900 m^3/d (Fig. 1).

When the Great Hanshin-Awaji Earthquake occurred in 1995, the Authority was unable to supply a sufficient amount of water because of significant damage to its water supply facilities. Therefore, the Authority established the Plan to Improve Earthquake Resistance of Facilities in 1996 to ensure that the facilities can reliably supply a sufficient amount of water even following an earthquake. It is also currently undertaking the renovation and seismic capacity improvement of individual facilities.

Meanwhile, in addition to maintaining and upgrading its facilities, the Authority has produced an Outline of Accident Preparedness Plan, which states appropriate measures and action to be taken by staff in emergencies such as natural disasters, accidents at water sources that affect water quality, and failures of equipment. We have added manuals to promote preparedness for individual crises and have taken measures to prevent sabotage of the water supply [1].

However, the most recent manuals and measures regarding the facilities were considered insufficient to deal with the potential damage that could be caused by recent abnormal weather and man-made threats such as acts of terrorism. Therefore, the Risk and Crisis Management Plan was developed through a planning process using a workshop method with the participation of the Authority's staff.

In this report, we mainly discuss the measures and actions to deal with natural disasters including earthquakes that are stated in the Risk and Crisis Management Plan. We also report its relationship with the current Facilities Improvement Plan and the advantages of adopting participatory planning using a workshop method to develop the Risk and Crisis Management Plan.

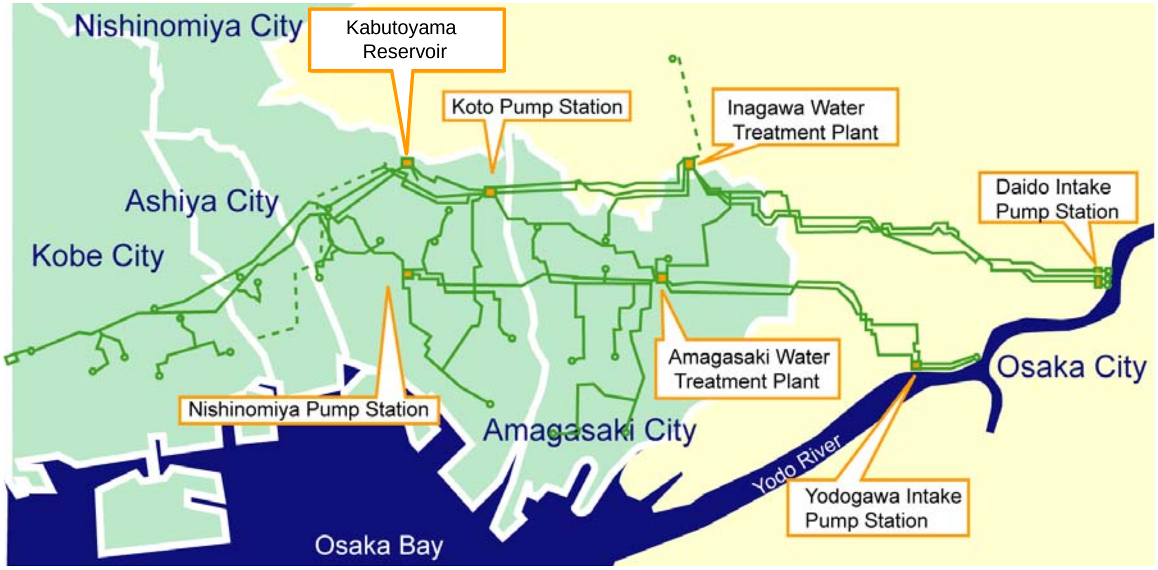


Figure 1. Outline of facilities of Hanshin Water Supply Authority

PREVIOUS RISK AND CRISIS MANAGEMENT

Since the Great Hanshin-Awaji Earthquake (1995), the Hanshin Water Supply Authority has implemented the following measures and renovation projects centering around the main facilities to reduce the damage and aftereffects of natural disasters.

- Renovation of water treatment plants and pumping stations to ensure that they can withstand level II earthquake motion
- Installation of anti-seismic pipelines with S- or NS-shaped joints
- Adoption of cogeneration system to reduce the risk of power failures caused by disasters
- Arrangement of reciprocal connection pipelines to receive water from other water supply authorities in an emergency

Moreover, the system for monitoring water quality is being enhanced and the following measures have been implemented to prevent man-made threats since the sarin gas attack on the Tokyo subway (1995).

- Installation of automatic security systems in main facilities
- Implementation of staffed security systems in water treatment plants
- Introduction of bioassay in intake pumping stations and water treatment plants

In addition to the above on-site measures and the enhancement of the monitoring system, we have prepared the following manuals so that staff can take prompt action in a crisis.

- Outline of Accident Preparedness Plan and Guidelines for Enforcement of Command in Emergencies (produced in 1994)
- Manual on Cryptosporidium Preparedness (produced in 1997)
- Manual on Disaster Preparedness (edition on preparedness for seismic disaster) (produced in 2000)

Furthermore, training program for natural disaster has been provided for all staff of the Authority, through which the validity of the manuals has been verified. At the same time, an initial response system has been established to ensure communication among staff in the case of a disaster.

DEVELOPMENT OF RISK AND CRISIS MANAGEMENT PLAN

Intensification of risk and crisis management

To date, the Authority has implemented measures to deal with crises such as earthquakes, power failures, and water leakage from pipelines; however, the probability of encountering crises has recently been increasing because of the severity of recent natural disasters, which may be partly caused by climate change, acts of terrorism, which are prevalent around the world, breakdown of information systems due to cyberattacks, and other factors. Accordingly, we decided to develop the Risk and Crisis Management Plan, as well as continue the ongoing improvement of facilities, so that the facilities will be able to rapidly return to normal operation even if an unprecedented crisis occurs in the future.

Risk and Crisis Management Plan

The Risk and Crisis Management Plan (hereafter, the Plan) provides general principles of risk and crisis management by assuming various crises including unprecedented crises, rather than focusing on individual crises. The Authority has prepared manuals on preparedness for individual crises on the basis of the Plan. These manuals are designed to allow future revision even if the circumstances surrounding the Authority change in the future and enable the systematic conduction of risk and crisis management.

Moreover, it is important that individual staff members participate in providing ideas and developing the Plan; therefore, another of our aims was to increase staff awareness of potential crises during the development of the Plan.

Development of Risk and Crisis Management Plan using workshop method

The Plan was developed using the framework of a strategic plan, which is a comprehensive plan clearly indicating how to achieve the mission statement, vision, and objectives of the Authority. More specifically, a strategic plan clearly shows the relationships between the ends and means, i.e., the goal, objectives to achieve the goal, policies/measures to meet the objectives, and projects/actions to implement the policies/measures [2], as shown in Fig. 2.

It is vital that the staff members concerned are involved in thinking about what measures are required to achieve the designated goal. For this reason, we developed the Plan by participatory planning using a workshop method.

A total of 35 staff members (13% of the entire staff) were selected from among engineering and administrative officials of each section of the Authority, and were divided into five groups to participate in a workshop (WS). First, a consensus was reached among the participants that "to provide a secure and reliable water supply in any crisis or emergency" was the ultimate goal to be achieved. Then, the problems that the Authority would have to overcome to achieve the goal were specified. Next, the participants discussed how the risk of these potential problems can be reduced and what the Authority must do to overcome them. In concrete terms, the following procedures were repeated during the WS: 1) the extraction of ideas through writing on cards, 2) building a structured system using the extracted ideas within the framework of a strategic plan, 3) deciding whether the structured system can be adopted as part of the Plan, and 4) obtaining a consensus among the WS participants. The above four processes were repeated [3].

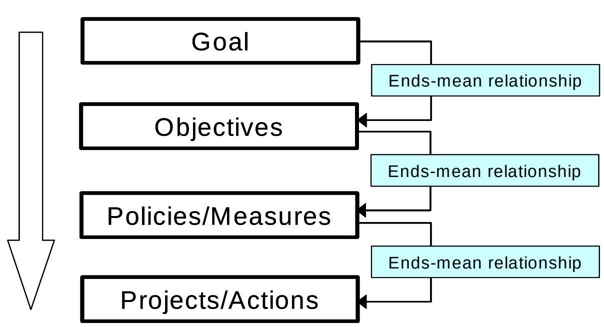


Figure 2. Structure of strategic plan

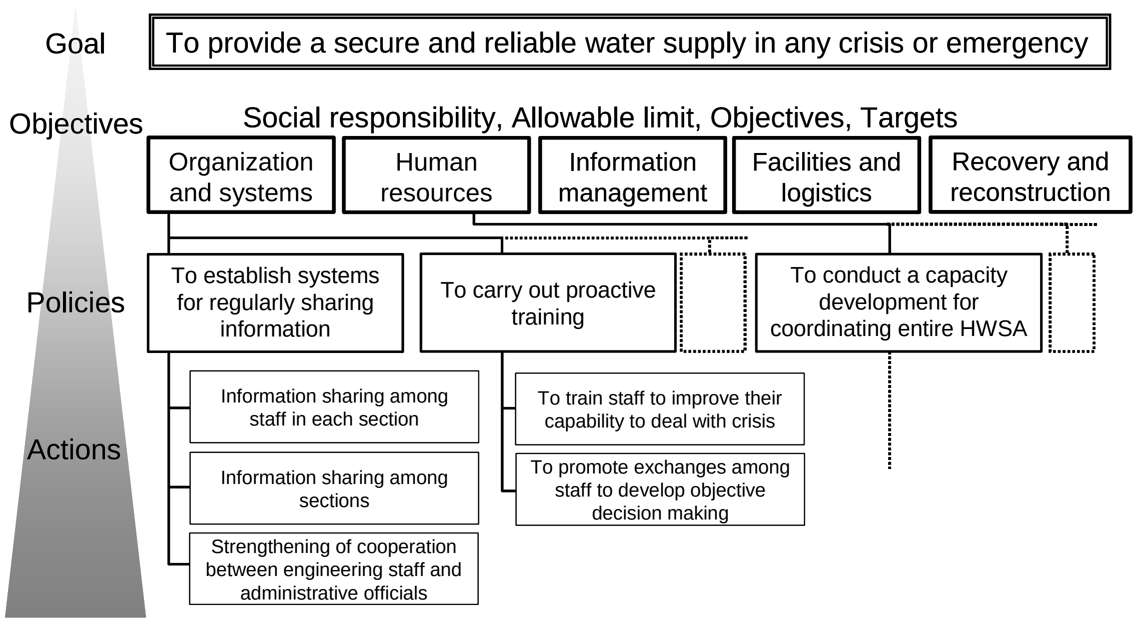


Figure 3. Framework of Risk and Crisis Management Plan

Setting of objectives

The participants selected appropriate idea cards regarding what must be done to achieve the goal of providing a secure and reliable water supply in any crisis or emergency, following the above-described procedures, then built a structured system using the extracted ideas by the affinity diagram method in accordance with the structure of strategic plans. As a result, the idea cards were classified into the following five categories: (1) organization and systems, (2) human resources, (3) information management, (4) facilities and logistics, and (5) recovery and reconstruction. Objectives were set for each of these five categories. Figure 3 shows the framework of the Plan.

Table 1. Part of Risk and Crisis Management Plan

Goal	Category	Objectives	Policies/Measures	Projects/ Actions	Activities	Operational procedures	Period
To provide a secure and reliable water supply in any crisis or emergency	Organization and systems	To establish an organization and system to ensure high awareness of possible crisis for all staff	Clarify role of each person in an emergency Establish system to share information at any time Establish contact system in an emergency, etc.				
	Human resources	To ensure that staff can give appropriate advice to directors and support workers in an emergency	Conduct a capacity development for coordinating entire HWSA Recruit excellent external human resources, etc.				
	Information management	To control information so that staff can make rapid and appropriate judgments and take action quickly recover water supply	Establish procedures for contact Assign roles of information collection in an emergency Provide information to related organizations, etc.				
	Facilities and logistics	To maintain facilities and logistics with high robustness to crises	Ensure water quality even in an emergency Carry out seismic capacity improvement and seepage prevention measures Realize preventive facilities for crises, etc.				
	Recovery and reconstruction	To ensure same understanding of crises between the HWSA and external organizations and carry out recovery in accordance with designated goals	Set goals for recovery and reconstruction Determine crisis level, etc.				

Development of strategic plans

To attain each objective, policies/measures, which are located below the objective in the structure, were discussed. To realize a higher level objective, the lower level tasks including projects/actions were systematically organized. Moreover, we added two steps, activities and operational procedures, below the actions to prepare manuals. Furthermore, the period of each activity and operational procedure was also examined to ensure its effectiveness. Table 1 shows a part of the Plan.

EFFORTS FOR NATURAL DISASTERS INCLUDING EARTHQUAKES

Policies/measures for each category of the Plan

The reasons for developing the Plan are to indicate the general principle of risk and crisis management with the assumption that even unprecedented crises may occur and to systematically prepare manuals for individual crises. When major policies/measures for natural disasters including earthquakes were extracted from the Plan prepared by the above procedures for each of the five categories, the following policies/measures were obtained for each category.

1. Organization and systems
   To establish a support system and to clarify the role of each person in an emergency
2. Human resources
   To conduct a capacity development for coordinating the entire Authority
3. Information management
   To establish a communication network and to offer information to related organizations
4. Facilities and logistics
   To improve the facilities for ensuring the quality and amount of water supplied
5. Recovery and reconstruction
   To set goals for recovery and reconstruction

Among these five policies/measures, the establishment of a support system in an emergency and the sharing of information can be incorporated in common manuals for individual crises. However, the policies/measures for facilities and logistics are generally related to the improvement of facilities, such as seismic capacity improvement. Therefore, we had to decide where the improvement of facilities as part of risk and crisis management should be positioned in the current Facilities Improvement Plan.

Improvement of facilities included in the Plan

The policies/measures for facilities and logistics mainly concern the improvement of facilities, which is generally planned to be implemented in the medium to long term. To ensure preparedness for natural disasters, the facilities should be improved while mainly focusing on seismic capacity improvement.

The Plan could assume that the control of the water supply in an emergency on the basis of the practical experience of the staff members in their respective sections. According to the opinions mainly provided by the staff of water treatment plants and pumping stations, the facilities must be improved to avoid total interruption of the water supply from the Authority in a crisis. Specifically, the amount of water treated and supplied in a crisis is assumed to be one of three levels: 75%, 50%, or 25%. This is because, as shown in Fig. 4, the water treatment facilities of the Authority, Inagawa

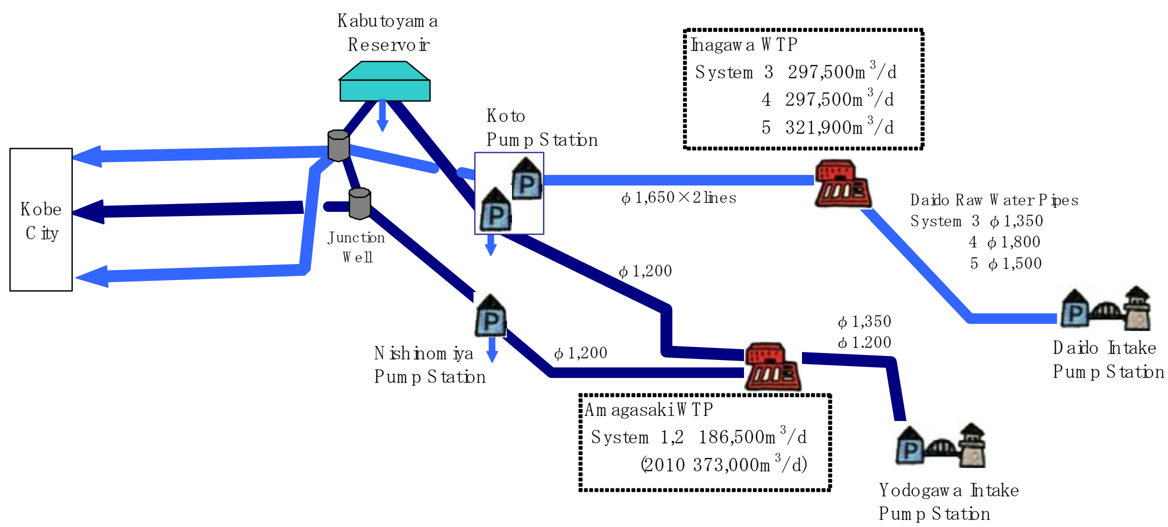


Figure 4. Capacities of HWSA facilities

Table 2. Part of the Plan for the category of facilities and logistics

Objectives	Policies/ Measures	Projects/ Actions	Activities	Operational procedures	Period
To maintain facilities and logistics to ensure resistance to damage	Ensure 75% of water supply	Take measures against power failures			Short term
		Promote seepage prevention	Take seepage prevention measures for power-receiving equipment		Short term
			Take seepage prevention measures for pumps		Short term
		Carry out seismic capacity improvement of facilities and pipelines	Evaluate earthquake resistance of facilities	Investigate soil conditions	Short term
			Promote antiseismic project	Renovate pipelines of systems 1, 3, and 4	Medium tern
				Complete construction of Amagasaki WTP	Short term
	Ensure 50% of water supply	Take measures against power failures			Short term
		Promote seepage prevention	Take seepage prevention measures for power-receiving equipment		Short term
			Take seepage prevention measures for pumps		Short term
		Carry out seismic capacity improvement of facilities and pipelines	Evaluate earthquake resistance of facilities	Investigate soil conditions	Short term
			Promote antiseismic project	Renovate pipelines of systems 1, 3, and 4	Medium tern
				Complete construction of Amagasaki WTP	Short term
	Ensure 25% of water supply	Take measures against power failures			Short term
		Promote seepage prevention	Take seepage prevention measures for power-receiving equipment		Short term
			Take seepage prevention measures for pumps		Short term
		Carry out seismic capacity improvement of facilities and pipelines	Evaluate earthquake resistance of facilities	Investigate soil conditions	Short term
			Promote antiseismic project	Renovate pipelines of systems 1, 3, and 4	Medium tern
				Complete construction of Amagasaki WTP	Short term
		Improve regulating reservoirs	Install more regulating reservoirs		Long term
	Ensure 75% of water treatment	Take measures against power failures			Short term
		Take seepage prevention measures for Daido Intake Pump Station	Take seepage prevention measures for power-receiving equipment		Short term
			Take seepage prevention measures for pumps		Short term
		Improve systems 3 and 4 of Daido raw water			Medium tern
		Complete construction of Amagasaki WTP			Short term
		Renovate systems 3 and 4 of Inagawa WTP	Carry out seismic retrofitting of water treatment plants		Medium tern
			Improve clear water reservoirs		Medium tern

WTP (916,900 m^3/d) and Amagasaki WTP (373,000 m^3/d), are roughly divided into four systems, each of which has a capacity of 300,000 m^3/d. Thus, the amount of water supplied is assumed to be a multiple of 25%, i.e., at least one system is in operation in a crisis. To this end, it is necessary to complete the construction of Amagasaki WTP to its full designed capacity and to carry out seismic capacity improvement of systems 3 and 4 of the Daido raw water pipes and Inagawa WTP. Table 2 shows part of the Plan for the category of facilities and logistics.

Reduction on Earthquake Vulnerability of Facilities

The Authority is carrying out renovation and seismic capacity improvement of individual facilities in accordance with the Plan to Improve Earthquake Resistance of Facilities (produced in 1996). The facilities to be renovated are prioritized to ensure the supply of at least the minimum amount of water necessary during reconstruction, taking into account not only the aging of facilities but also the control of the water supply during reconstruction. In addition, the Facilities Improvement Plan, the draft of which was mainly drawn up by the Planning Section, is regularly reviewed in accordance with the financial plan produced every four years.

Policy for improvement of facilities

In the improvement of facilities, the Authority is promoting not only structural reinforcement of each facility but also the improvement of the earthquake resistance of the entire system from the viewpoints of security of redundancy and fail-safe operation. Also, a reserve capacity is ensured during emergency operation to increase the stability of the water supply.

Specifically, infrastructure such as water treatment plants and pumping stations is renovated and reinforced to a strength sufficient to withstand expected earthquakes to ensure continued supply of the required amount of water at and after an earthquake occurs. Moreover, the Authority aims to ensure the necessary capacity of facilities including a reserve capacity in regulating reservoirs.

Table 3. Goal and each step of the Plan to Improve Earthquake Resistance of Facilities

Period	Policy of improvement	Water treatment plant			Main pipeline (raw water pipes and water transmission pipes)		
		Anti seismic level (%)	Anti seismic facilities (m^3/d)	Entire facilities (m^3/d)	Anti seismic level (%)	Length of anti-seismic pipelines (km)	Total length of pipelines (km)
1994	-	8	80,000	1,048,000	29	35	123
First step (1995-2000)	･Seismic capacity improvement of damaged and aged facilities ･Renovation of Amagasaki WTP (373,000m^3/d)	54	694,900	1,289,900	47	61	131
Second step (2001-2010)	･Seismic capacity improvement of systems 3 and 4 of Inagawa WTP and Daido raw water pipes (595,000m^3/d) ･ Improvement of reserve capacity of purified water	100	1,289,900	1,289,900	58	75	131
Third step (2011-)	･Establishment of wide-area network	-	-	-	-	-	-

To improve the earthquake resistance of pipelines, the thorough renovation of aged pipelines is necessary. In addition, the multiplexing of water transportation using connection pipelines is promoted to increase the reliability of the network of facilities.

Furthermore, measures are being taken to ensure the water supply even in times of disaster by adopting a cogeneration system using natural gas, which aims to reduce the vulnerability to energy shortages in times of disaster, and by arranging the facilities to utilize the potential energy of water in regulating reservoirs.

The process of the improvement of facilities roughly comprises three steps. Table 3 summarizes the policy and objectives of the improvement plan.

Improvement of facilities and risk and crisis management

Currently, the improvement of facilities is being carried out in accordance with the Plan to Improve Earthquake Resistance of Facilities.

(Amagasaki Water Treatment Plant)

Initially, the construction of Amagasaki WTP was to be completed in 2000 to provide a total capacity of 373,000 m^3/d; however, only half the capacity, 186,500 m^3/d, has been achieved because of the lower than expected demand for water. Subsequently, the full implementation of the original plan was agreed upon with the aim of ensuring a balanced amount of water to be treated by the water treatment plants of the Authority and ensuring sufficient water supply during the renovation of facilities. This improvement is scheduled to be completed in 2010. Therefore, this is, in practice, considered as second step improvement although it was designated as first step improvement in the Plan to Improve Earthquake Resistance of Facilities.

(Seismic capacity improvement of systems 3 and 4)

In the Plan to Improve Earthquake Resistance of Facilities, the seismic capacity improvement of systems 3 and 4 of Inagawa WTP and the Daido raw water pipes was scheduled as second step improvement. Improvement is currently planned to begin after 2010.

To confirm the progress of seismic capacity improvement being carried out in accordance with the Plan to Improve Earthquake Resistance of Facilities, the degrees of progress for water treatment

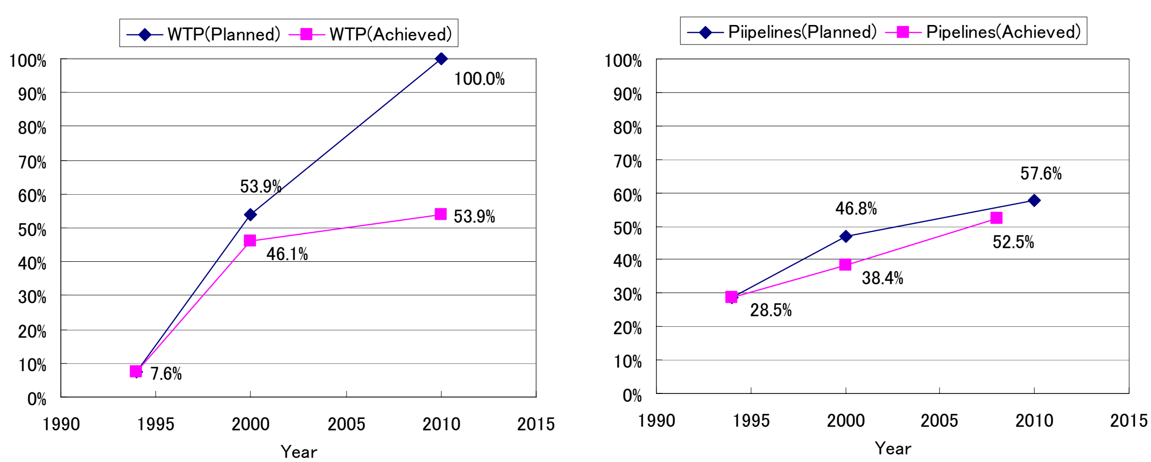


Figure 5. Anti-seismic level of WTP and pipelines

plants and the main pipelines (hereafter, anti-seismic level) are expressed as percentages, as shown in Figure 5. From the anti-seismic level, it can be seen that the actual improvement of facilities is behind schedule because of the lower than expected demand for water and unfavorable financial conditions. The anti-seismic level for water treatment plants is 54% at present, mainly because of the delay in the seismic capacity improvement of Inagawa WTP. The low anti-seismic level of the main pipelines is due to the delay in the seismic capacity improvement of the Daido raw water pipes.

As described above, in the improvement of the facilities determined in the Plan, it is vital to avoid a complete cessation of the operation of all the facilities of the Authority. To ensure a sufficient amount of water supply for each system, the Authority must continue to carry out improvement of Amagasaki WTP to achieve its full designed capacity and the seismic capacity improvement of Inagawa WTP and the Daido raw water pipes. Also, in terms of risk and crisis management, the improvement of facilities, which is now behind schedule, must be carried out promptly.

Therefore, we must improve the capability of staff and facilities to deal with crises by preparing manuals in accordance with the Plan to compensate for delay to the improvement work of facilities as part of risk and crisis management.

Preparation of manuals and training

Many of the actions for dealing with crises are concretely described in the Plan as activities and operational procedures, and we have prepared manuals on the management of individual crises referring to these actions. Figure 6 shows the procedure for preparing such manuals. Specific crises are assumed and appropriate measures to deal with them are extracted from the activities and operational procedures in the Plan then arranged to form manuals. Required information and related materials for emergency response are specified, which are presented in the manuals in the predetermined format.

To verify the validity of the manuals prepared by the above procedures, the Authority has implemented field training and tabletop exercises following these manuals. The field training is carried out after informing the participants of the scenario. The simulation and actual training results are compared for evaluation. In post-training review meetings, the participants can discuss the

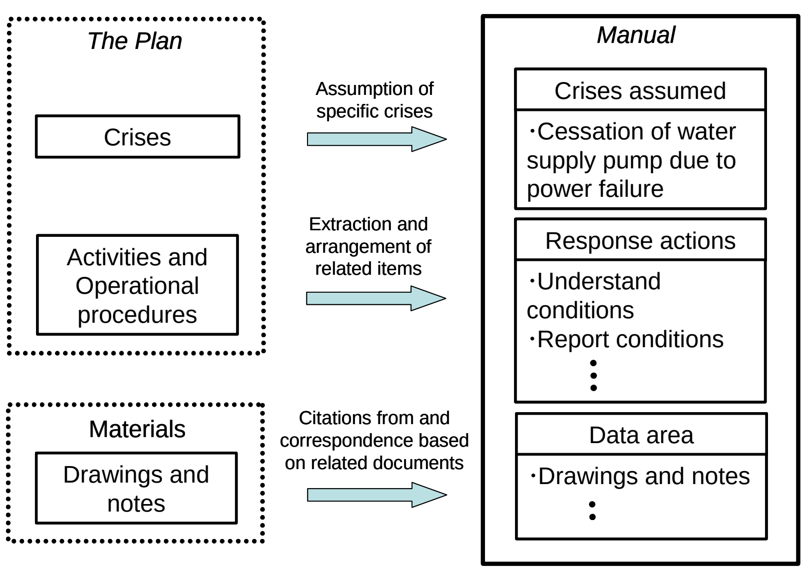


Figure 6. Procedure for preparing manuals

Table 4. Capacity development program of field training and tabletop exercises performed

Year	Contents of training/exercises	Type
2009	WS on running-water operation, 4cases	WS
2008	Treatment of leakage (considering effects on trains)	Tabletop exercise
	Power failure at WTP	Tabletop exercise
	Detection of abnormal quality of raw water at intake pump station (caused by oil spillage)	Tabletop exercise
	Call-up training	Training on communication
	WS on response to accidents, 2cases	WS
	WS on running-water operation, 2cases	WS
2007	Detection of abnormal quality of raw water at intake pump station (caused by contamination by poison)	Tabletop exercise
	Treatment of leakage (considering accidents causing injury)	Tabletop exercise
	Training for dispatching support staff (emergency supply of water)	Field training
	WS on case studies, 17cases	WS

problems of the current training methods, and the appropriateness of the contents and procedures of the training, enabling the evaluation of the manuals. In the tabletop exercise, a group of several participants focus on a case involving an accident and discuss the problems and solutions related to the case. Training based on the assumption of various situations in the absence of leaders is also carried out for a limited number of participants because we cannot always deal with a crisis under ideal staffing conditions.

In practice, discussion WSs are held among relevant staff members before performing operations with a high risk of causing an accident and after dealing with equipment failures with the aim of identifying problems related to the operations, elucidating the causes of the accidents, and preventing their re-occurrence. Thus, as concerned parties, it is indispensable to increase the awareness of potential crises as well as improve the problem-solving ability of staff members. Table 4 shows the capacity development program of field training and tabletop exercises performed since the establishment of the Plan, and Fig. 7 shows an operation in field training and a discussion in tabletop exercise.

As mentioned above, manuals are prepared by participatory planning using a workshop method on the basis of the Plan, and tabletop exercises and field training are carried out in accordance with the manuals, through which the manuals can be reviewed and improved. Moreover, the preparation

Figure 7. Operation in field training and discussion in tabletop exercise

and improvement of manuals is equivalent to the updating of the Plan because it was comprehensively and systematically developed using the framework of a strategic plan and thus includes the measures and actions required to attain the goal of providing a secure and reliable water supply in any crisis or emergency. That is, operating a plan-do-check-action (PDCA) cycle, which comprises preparation and improvement of manuals, tabletop exercises and field training, post-training review meetings, scrutiny of the manuals, and the update of the Plan, with the participation of staff on the basis of the Plan, will lead to improved crisis management ability of the staff.

CONCLUSION

We developed the Risk and Crisis Management Plan by participatory planning using a workshop method. The policies/measures to ensure preparedness for natural disasters including earthquakes have features in common with those for other crises; for example, the establishment of a support system and the development of information systems in the aftermath. It is point out that the mission of HWSA would require promptly implementation of these measures by preparing manuals and carrying out capacity development. On the other hand, it is necessary that vulnerability reduction of the facilities such as seismic capacity improvement is being carried out as a medium to long term project because of the delay in completing the project under the current Facilities Improvement Plan. However, this must be advanced in a sensible manner by taking financial conditions into consideration.

Moreover, manuals for individual crises have been prepared on the basis of the Plan, and capacity development program has been carried out using the manuals. Owing to the participation of the staff in the preparation of the Plan and the manuals, the opportunity of training being carried out, such as tabletop exercises in small groups, is increasing. This procedure would encourage the staff to learn about risk and crisis management and acquire an increased awareness of potential crises, leading to an improvement in their capability of dealing with a crisis. To compensate for the setbacks to risk and crisis management due to the delay in the completion of the improvement of facilities, it is necessary to intensify the implementation of measures to improve risk and crisis management by establishing and updating of manuals and carrying out capacity development with the operation of PDCA cycle.

REFERENCES

[1] Okamoto, T., Kobayashi, K., Komiyama, K. and Mihara, M. 2006. “Program for Risk and Crisis Management on Hanshin Water Supply Authority,” Proceedings of 50th Annual Conference on Kansai Branch JWWA, pp.176-178.

[2] Tamura, K., Hayashi, H., Tatsuki, S., Maki, N., Tanaka, S., Kondo, T., Horie, K., Banba, M., Karatani, Y., Hasegawa, K. and Fukasawa, Y. 2004. “Development of Participatory Strategic Planning in the Process of Disaster Reduction Planning: A Case Study in Marikina City, Philippines,” Journal of Social Safety Science, Institute of Social Safety Science, (6): 129-138.

[3] Hirayama, N., Hayashi, H., Itoh, S. 2007. “Participatory Planning in Development of Comprehensive Crisis Management Plan for Water Supply Authorities,” Proceedings of 5th AWWARF/JWWA Water System Seismic Conference, pp.167-178.

Earthquake Disaster Response Project for Water Facility at Yilan Area

Bing-Xin Wang[1], Zhi-Xian Lin, Jao-Shyan Chen and Yi-Lun Hou

ABSTRACT

The 921 earthquake had once devastated our water network facilities in Taiwan. Due to the fact that the Philippine Sea Plate would constantly collide with the northern sea plate, most of the earthquakes happen at the outer sea floor of Hualien and Yilan, and it occurs that the Eighth Branch of Taiwan Water Corporation is located at seismic sensitive are, and is confronted with such hazardous environment for water supply. Therefore, it is necessary that operation personnel needs to think of coping strategies against potential disasters caused by earthquakes. Currently, the eighth branch has aggressively facilitated to set up supervisory control and data acquisition (SCADA) and geographic information system (GIS), and they will be used to collect information from the changes of primary indicators of water networks and facilities such as water quality, water level, water pressure and water quantity. The collected data will be used to calculate possible situation and rage of damage caused by disaster in time. Then, patrol personnel will carry with them mobile device, their PDA being equipped with information of pipeline chart, basic information of facility, inspection list, and satellite positioning, to any possible facility of damage and check if any damage is done to water tank, and machine engineering equipment to see if it will compromise water supply and network pipeline, and if they could lead to leakage or machine failure. In the future, each mobile device will be integrated with GPRS so that site information can be feedback to the emergency center for its personnel to conduct analysis, helping them to bring forth decision making for emergency. Once the disaster is confirmed, the patrol personnel will be notified to carry out emergency administration at the site, and they will also change mode of water supply, depending on the range of disaster. At the same time, backup team for support will be dispatched for emergency repair. As communication technology and innovative skills are integrated into water supply industry, they will help reduce the needs for manpower and reduce the time for emergency repair and recovery so that the personnel can deal with their work in ease, largely enhancing the efficiency to respond to emergency situations.

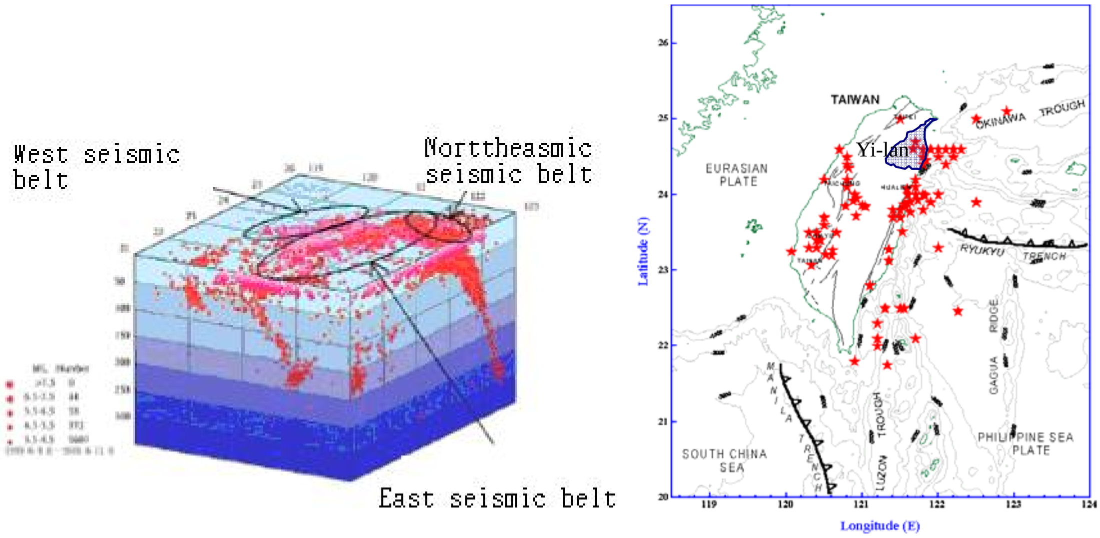
Keywords: Earthquake**,** Supervisory Control and Data Acquisition (SCADA)**,** Geographic Information System (GIS)

1. INTRODUCTION

Taiwan is located at the intersection of Eurasian plate and Pacific Philippine sea plate, considered as one of the areas with most frequent earthquakes. In 20th century, there were around 100 disasters triggered by earthquakes in Taiwan, please refer to chart 1 for detailed layout of earthquake areas, and these disasters have caused serious casualties and property damages. The most serious earthquake disaster was Chi-Chi earthquake that happened on 9/21 1999 at 1:47.159AM, and oil liquefaction was reported in many areas. Besides, countless of public and private buildings collapsed, with roads and communication channels being cut off, which increased difficulty for search and rescue. Yilan is located at the seismic

[1] Manager, Eighth Branch, Taiwan Water Corporation, 1200@mail.water.gov.tw.

zone of the north-east seismic zone at the eastern part of Taiwan. For the north- eastern seismic zone, it is affected by sea-floor spreading from Okinawa trough, which belongs to shallow seismic activity area as it is accompanied by geothermal and volcanic activities (around Turtle Island area). Because the Philippine plate is directly crushing the Eurasian plate at eastern seismic zone, the area is found with the most earthquake frequency. In addition, the Philippine plate is neighboring the east coast of Taiwan and extends from Taitung, Chenggong, Hualien to Yilan. Thus, 18 disastrous earthquakes happened in Yilan and its surrounding areas between 1901-2006, and the most serious disasters happened between 1922 to 1967. The average earthquake magnitude of the area is 6.4, resulting in total toll of death to 21, with 97 houses wholly devastated. In 1922 and 1987, they are the years with the most serious damages, and 24 houses and 21 houses were devastated respectively.

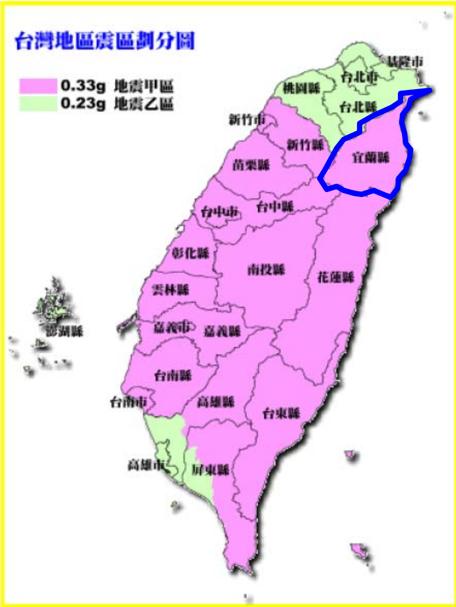


Left: Sensible quakes Right: Disastrous quakes

Chart 1 Earthquake layouts in Taiwan during 20^{th} century (1901-2000) (Source of information: website of Central Weather Bureau, Ministry of Communications and Transportation)

Life sustenance system primarily refers to facilities and pipelines of running water, sewage system, natural gas, gasoline, transportation, electricity, and telecom, which are found with complex and huge facilities and underground pipelines, being the infrastructures of a city. However, life sustenance system is fragile to earthquakes, and they are not easy to repair and recover right after disaster. Moreover, even if it’s only partially damaged, it can still devastate the entire system, resulting in the entire function of the area to shut down, which will further affect the social function of the devastated areas. At the same time, the wreckage of life sustenance system will not only inconvenience the people in the disaster area, it will also expend the disaster and stall rescue operations. For Example, insufficient supply of running water will cause trouble to put out fire, and can also bring about sanitation problems in the city, causing the widespread of epidemic. Next, the dysfunction of power and communication system will delay the report of disaster situation and coordination of rescue teams. Therefore, quake-proof and disaster countermeasure plan of life sustenance system as well as contingency plan of disaster prevention have become the necessary conditions to build up a safety and urban society.

After Chi Chi earthquake, the Construction and Planning agency, Ministry of the Interior has revised quake resistance design on Building Code and Regulations in December 1999, and areas in Taiwan are divided into class A and class B categories based on the level of corresponding acceleration coefficient as 0.33g and 0.23g. The earthquake areas include municipality, county, city, township and town as listed in diagram 2, and Yilan County is listed as class A earthquake area [2],



地震甲區：	
宜蘭縣、新竹市、新竹縣、苗栗縣、台中市、台中縣、彰化縣、南投縣、雲林縣、嘉義市、嘉義縣、台南市、台南縣、花蓮縣、台東縣。	
高雄縣	旗山鎮、三民鄉、六龜鄉、內門鄉、甲仙鄉、杉林鄉、美濃鄉、桃源鄉、茂林鄉。
屏東縣	恆春鎮、九如鄉、內埔鄉、里港鄉、車城鄉、牡丹鄉、長治鄉、來義鄉、泰武鄉、高樹鄉、春日鄉、獅子鄉、瑪家鄉、萬巒鄉、滿洲鄉、霧台鄉、鹽埔鄉、麟洛鄉、三地們鄉。

Chart 2 Earthquake area divisions for quake resistance design of buildings in Taiwan (Source: Construction and Planning Agency, Ministry of the Interior)

Earthquake is formed mainly when earth's crusts are pressed against one another, and the earth's surface is thus deformed and dislocated. If tap-water facility is close by faulting area, it will be damaged by landslide caused by an earthquake. Moreover, if the earth cracks open or land caves in it will also damage the water pipelines for supply to urban area. Strong quakes will, oftentimes, cause water hidden in the soil to leak out, causing the soil to liquefy and become fragile. Then, the foundation of drinking water facility or bridges holding the pipelines will loose support, and they might sink, tilt, collapse or break apart. Strong earthquake can directly destroy the pipelines, gas pipelines, and electricity cables, whereas leaking gas might cause fire hazard if it comes into contact with fire source or short circuit of electricity. Once such accident occurs, since most of the water pipes are been damaged, it would be difficult to put out fire. Worse yet, the structure of reservoir might rupture and collapse from the massive shaking of water and earth's surface or landslide, and the escaped water can cause flood and devastate the populated area downstream. Such destruction of life sustenance system mentioned above will bring about serious casualties and property damage. Therefore, effective countermeasures and plans should be put forth to fight against earthquake disasters especially for drinking water facilities at Yilan, helping with fast recovery of city and country after disaster.

2. DISASTER PREVENTION AND RESCUE OPERATION OF THE CENTRAL GOVERNMENT

"Earthquake Disaster Prevention-Rescue Operation Project" is the top priority rescue plan against earthquake disasters presented by Taiwan's National Disaster Prevention and Response Commission. This plan is found with six major chapters as general principle, disaster prevention, emergency response for disaster, post-recovery and restoration of disaster, tsunami disaster prevention and coping plan and project implementation, and supervision and evaluation. The following is the collections in relation to earthquake disaster prevention and rescue project for drinking water and life sustenance system:

1) Disaster prevention chapter:

The disaster reduction plan is used to ensure the preservation of functions of life sustenance system. Besides, disaster preparedness includes the establishment of response mechanism, collection of disaster intelligent, reconditioning of report and analysis application for disaster, preparation of fire extinguishing and emergency medical gears, providing drinking water and daily necessities, preparation of supplies, emergency recovery of facilities and equipment, and secondary disaster prevention. In the research for earthquake disaster prevention-rescue plane, it is to construct and share disaster prevention databases, and to study technologies and responses on the prevention and rescue for earthquake related disasters.

2) Emergency response for disaster

The following is the primary issues of discussion for the research.

A. In subject of ensuring the collection, reporting, and communication of disaster situation, when an earthquake happens and disasters occur, it is via the emergency contact information system so that information of casualties and disaster situation can be provided to rescue personnel to increase rescue chances. The public water company (public utility) will collect damage information on life sustenance system to commence emergency measures, and organize coping measures for the disaster. Then, it will review the disaster situation and emergency counter-measure so as to request for backups and recommend items for report. Also, information about the leaking of water from storage reservoir, weirs and other water related structures as well as the leaking of chemicals can be collected so that emergency and response measure can be requested.

B. For emergency response system, the timing for central disaster response center to be in operation is when the Central Weather Bureau announces an earthquake to be more than level 6, with the estimation of more than 15 people killed, injured, or missing, and the scenario of disaster is serious as many people are awaiting for rescue, being recognized acknowledged by the Ministry of the Interior. The timing for local disaster response center to be in operation is the same as that of central response center. The only difference is that local disaster center is in charged by the local government, and it will begin operation as approved and acknowledged by local government. After the local disaster response center is in operation, it should keep close contact with the central disaster response center. Besides, the response center is responsible to dispatch disaster site coordination personnel to report on responsive procedures for major disasters, and dispatch support personnel from the military.

C. For emergency response to earthquake disaster, public medical facilities within the disaster area should conduct emergency repairs to their buildings and medical equipment. When a conflagration occurs, it is necessary to decide what will be the most important parameter of defense, and request for assistances from the firefighting units. If necessary, help from the military can be requested. The primary emergency delivery to disaster area by the emergency response team from disaster prevention-rescue institution should include repair personnel and materials needed to insure the security of drinking water facility and for primacy of disaster response personnel. Then, secondary delivery should include drinking water and daily necessities to supply the needs.

D. With follow-up emergency measures, professionals and technical personnel from all fields will be dispatched or requisitioned, and they will inspect or evaluate the damage, slanting, and cracks on reservoir and public facilities as well as will perform emergency demolition, reinforcement, and set up alert zones based on their authority. If an area is alerted as a high-risk area, the related institution and residences should be notified on its own and warning and shelter facility be implemented to prevent secondary disaster. In order to prevent leakage of public hazardous materials and hazardous substances, institutes of public utilities should monitor factories and their surrounding facilities to prevent leakage of toxic chemical substances. Also, areas that are already under serous pollutions should be isolated and monitored, and environmental monitor should be conducted with countermeasures taken to prevent the pollution from spreading. Local environment protection bureau should intensify the examination on water quality after earthquakes. For emergency repairs and supplies of public water pipeline, water reservoir, and weirs, they should also be done quickly. Public utilities service should appreciate the needs of disaster victims, and should coordinate medias to pass down information about damage and restoration on life sustenance system and public facilities as well as government policies taken in response to the disaster.

3) Recovery after the disaster

This area states basic direction for recovery and rebuilding of the disaster area; in area of emergency recovery, it states about the fast repair and recovery for damaged structures with streamlined procedures. At the end, it is the project for redevelopment planning and support and maintenance for victims in the disaster area.

3. CURRENT EMERGENCY MEASURE OF THE HEADQUARTERS AND DISTRICT OFFICES

Yilan's drinking water facility is managed by the eighth district, and the unit is found under Taiwan Water Corporation. For response to various emergencies, Taiwan Water Corporation will handle them based on "Operation outlines of disaster emergency response team." The main purpose of the outlines is to organize operations for disaster emergency measures such as command, supervision, coordination, data collection, and data transfer during disasters and emergencies. Taiwan Water Corporation's emergency response team will assemble at the time when Central Disaster Response Center or when the Ministry of Economic Affairs disaster emergency response team is assembled, or when there is a need to supply water. The competence authority will be water supply department at the headquarters, while it is the engineering unit of the eighth district. The duties of unit are listed as follows:

1) Convene meeting of emergency response team to increase the speed handling the situation. Then, senior level stuffs will be dispatched to the company to supervise and facilitate emergency measures depending on the disaster situation.
2) Execute the company's disaster response duties, and cooperate with the Central Disaster Response Center and Ministry of Economic Affairs disaster emergency response team to handle safety measures related to the disaster.

3) Collect information related to the damages for water supply system in each district, and report the information to the Central Disaster Response Center, the Ministry of Economic Affairs disaster emergency response team and other related organizations.
4) Command and supervise repair and rescue duties for water supplying facilities.
5) Inspect and supervise the emergency repair, rescue, and reconstruction projects.
6) Supervise examination of drinking water quality .
7) Supervise the advocacy of water conservation during time of disaster.
8) After periodical mission of the team is dismissed, following measures for subsequent recovery works are turned over to the company's competent divisions. The organization is installed with a team convener, a vice convener, and an executive secretary. Also there are a few emergency response supervisors in each important water supply area. The convener will dispatch and divide the group based on mission needs into water supply division, construction division, material division, water quality division, operation division, administration division, accounting division, human resources division, evaluation division, information division, safety division, ethics division, management office in each area, and engineering works office in each area. The following is the list of duties for each local management office:
   a) Responsible for rescue, emergency repair, water supply, and cleaning up.
   b) Execute missions from the disaster emergency response team of the headquarter, and the information gained during the operation is reported around the clock.
   c) Provide manpower to support each water supply and water purification facilities during typhoon period.

   The operation is divided into 3 levels based on news as the Central Weather Bureau announces the intensity of typhoon and earthquake has reached level 6 and also torrential rain, or when it is determined by the water supply division it needs to start operation, while related units of residences have, with report from water supply division and agreement from the convener, been invited.

To reinforce disaster against disaster prevention, the management office in each area should perform disaster simulation exercise based on emergency incident of water supply that had occurred or could have occurred. Also, to inculcate professional knowledge into company's employees and increase intensify the result of training each management center will should disaster prevention educational seminar for emergency and rescue according to the classification of emergency water supply at each facility. The classification of disasters and emergency incident report category for the company and its affiliated institutes are is shown on Table 1.

Table 1 Level classification table on reports of various disaster and emergency events by the Taiwan Water Corporation and its affiliated authorities .(Source: Taiwan Water Corporation)

Types of notification	Items
Class A Scenario	1.suspension of water suppler for over 48 hours and with 200,000 households affected as caused by disaster.
	2.local water reservoir or dam is damaged and might rupture or the water source is polluted, which might affect the life of people or affect the businesses' operations.
	3.Deficiency of water supply for over 30%.
	4.Incident of industry safety and hygiene that causes more than 10 casualties or missing people.
	5.daily water consumption over 50,000 cubic meters with water turbidity over 6000NTU
	6.18 hours after the Central Weather Bureau has announced a land and sea typhoon warning.

	7.After the Central Weather Bureau has announced torrential rain warning with accumulated rainfall in excess of 350mm per day, and the company considers it necessary to notify people concerned
Class B Scenario	1.Over 50,000 households are affected for 24 hours without running water caused by disasters.
	2.Deficiency of water supply for over 20% to 30%
	3.local water reservoir or dam is damaged and might rupture or water source is polluted
	4.A incident of industry safety and hygiene occurs, leading to more than 5 deaths or missing people.
	5.Major safety incident occurs or when public petition occurs, turning into a violent incident.
	6.When employees organize a demonstration or a strike that affects the operation.
	7.The daily water consumption rate is over 50,000 cubic meters with water turbidity over 3000NTU
	8. 12 hours after the Central Weather Bureau has announced a land- and sea typhoon warning.
	9.After the Central Weather Bureau has announced torrential rain warning with accumulation of rainfall in excess 200 mm per day, and the company considers it necessary to notify people concerned
Class C Scenario	1.Over 1,000 households are affected for 24 hours without running water caused by disasters.
	2.Over 350mm of rupture found on water pipeline.
	3.An incident of industry safety and hygiene occurs and it has caused 3 or more deaths or 1 or above injuries.
	4.An earthquake above 4 magnitude (including) occurs in the area of jurisdiction
	5.After the Central Weather Bureau has announced torrential rain warning with accumulation of rainfall in excess of 130 mm per day, and the company considers it to notify people concerned
	6.When the Central Weather Bureau has announced sea and -land typhoon warning.
	7.Not yet reached class B scenario, the situation is under controlled and will not deteriorate.
Media Reports	If any news media bring out negative report, it might trigger wide attention and affect the company image.

Because the drinking water facilities are different in various service areas within each management center, each center has its own plan for emergency response measure. For Yilan area, it is based on the “disaster emergency response plan of eighth branch management office. When the area is facing draught or natural disasters such as typhoon, torrential rain and such that will cause water supply inefficiency or damage to the drinking water facilities in short span of time, to maintain primary water supply, night time water decompression, water supply regulation, and water rationing will be implemented based, and the use of reserve water and allocation of water from water sources in neighboring area are implemented to provide emergency water supply. The eighth branch management office disaster emergency response center is established whenever it is needed, and the .team and its job structure for emergency response team are shown on table 2. Their main duty is to perform delivery of emergency water and perform rescue operations.

The eighth branch management office is located at Yilan territory, and it is an independent water supply area that is unable to obtain water supply from neighboring branch management offices. Within its area of service, there are 10 water supply systems, and it is combined by 2 supportable water supply systems from Yilan City and Loudong Township, with8 small independent water supply systems in Songluo, Yingshi, Sihji, Nashan, Dongao, Nan-ao, Jinyang, and Yingshi. In addition, there are 6 water supply systems with dual water sources, and when water sources are depleted, underground water will support water needs. There are 12 water service areas in the territory including Yilan City, Loudong Township, Toucheng Township, Suao Township, Jiaosi Township, Chuangwei Township, Yuanshan Township, Sansing Township, Datong Township, Dongshan Township, Wujie Townshipm and Nanao Township. In total, there are 140,000 household users, and water supply every day runs out about 160,000 cubic meters. The water capacity runs up to 225,510CMD, with water supply rate at 91%.

Table 2 Emergency response team structure and job table of the Eight Branch

Convener : manager — Comprehensive supervision on emergency response team' s response operation and responsible to report the response procedures to the headquarter.

Vice convener : secretary — Assist in supervising emergency response team' s response operation and distribute news announcements.

- water quality division — Perform water quality examination and gather and organizeinformation during disaster period
- securitydivision — Handle the protests from the public and handle the safety of the facility and support the police.
- accounting division — Responsible for funding used in confirming the constructions and reconstructions and other accounting duties.
- administration materials division — The distribution, controlling, operation of related materials equipment used for repairing. Handle the funding, vehicle distribution and general administrative duties. Support andsupervise the distribution of daily necessities. Responsible in supplying materials needed for repairs.
- engineering worksDivision — Responsible in maintaining normal water supply during disaster. Responsible in Inspect operations, command and coordinate, recovery and repairs.

Chart 3 Chart of water supply areas

The eighth branch management office divides disasters cause by earthquake into strong shock damage, pipeline rupture, low water pressure, water quality abnormity, power failure and leakage of sodium hypochlorite, and protective measures and rescue operation are elaborated as follows:

1) Emergency incident caused by strong shock.

A. Protective operation

(A).New buildings should use damage coefficient set up after 921 earthquake as reference on the structures to insure the safety of equipments.

(B).pipelines going through bridges, walls, and other areas that are easy to be dislocated should use bypass holes to connect the joints of the water pipes to increase quake resistance coefficient.

(C).Higher quality pipes with higher flexibility should be used for new or replacing old pipes.

(D).District water supply and system network should be maintained and surveyed periodically.

(E).Prepare enough backup materials in storage.

(F).Examine safety and maintenance situation on the structures, joints, and foundation slopes on the water collectors, water purifiers, and water distributors annually .

B. Emergency rescue operation

(A).Process emergency response process based on the division of labor.

(B).Investigate the amount of damage caused by earthquake in the area and provide recovery plan.

(C).Call for related companies to support rescue operation.

(D). If the rescue operation become inefficient and water supply is becoming depleted, it should be reported and take response measures as regulated.

2) Emergency incident of leaking and rupturing pipelines.

A. Ordinary prevention operation:

(A).Examine and repair pipeline network and venting equipment annually, and operate the valve according to regulation to prevent gas storage or gas explosion that might damage the pipelines.

(B).Priority evaluations should be taken on important roads to decide the operation of adding new lines or replacing old lines.

(C). Organize the schedules for reserve personnel and make agreements with companies to conduct emergency construction.

B. Emergency rescue operation

(A) Process emergency respond operation based on the division of labor.

(B) Investigate the amount of damage in the area and provide recovery plan.

(C) Provide safety and protection measures at the location with ruptured pipeline, and shut down valve to reduce water leakage.

(D). Call for related companies to support the rescue operation.

(E). If the rescue operation become inefficient and water supply is becoming depleted, it should be reported andtake response measures as regulate.

(F) If necessary, provide water trucks to deliver water to the needed location.

3) Emergency incident of low water pressure.

A. Ordinary prevention operation:

(A) Set up cross water purification and water supply system support mechanism; maintain good cross-system support pipelines; drain water periodically and maintain pipelines' usability. Above ground and underground water sources should be maintained to complement each other.

(B) District water supply and system network should be maintained and surveyed periodically.

B. Emergency rescue operation

(A). Process emergency respond operation based on the division of labor.

(B). Investigate the problem with the insufficiency of water supply and coordinate with the public water support.
(C). Call for related companies to support the rescue operation.
(D)If the rescue operation become inefficient and water supply is becoming depleted, it should be reported and take response measures as regulated.
(E). If necessary, implement district water rationing or provide water trucks to deliver water to the needed location.
(F) The pipeline should be cleaned through draining water after repair to prevent malfunction at terminal end and trigger siphon effect, which will cause secondary pollution to water supply.

4) emergency incident of Water quality :

A. Pollution source.

(A). External pollution source: pollution from the polluted water, polluted oil, chemical waste, as well as domestic sewage, agricultural pollution, fish poisoning, garbage dumping, and slope collapse that runs into water gathering territory caused by earthquake and causing high turbidity of raw water or damage to water quality.
(B). Internal pollution: pollution from equipment malfunction causing overdose or insufficient disinfectant and the leakage of oil by earthquake.

B. Ordinary prevention operation:

(A). Appoint special personnel to patrol and take record of water gathering area periodically, and report should be made if the area appears to be polluted.
(B) Install leakage prevention system and seal it with manhole covered on the water collection area and wells.
(C) Install fail-safe mechanism with equipment at the stations to prevent secondary disaster.
(D) Prevent water from possible pollution sources during regular days as well as observe and perform exercises. The precautionary alert will indicate pollution source by the water extraction area, and one can also observe abnormality of fish activities in the fish tanks, oil smells from the fresh water, abnormality in the ph conductivity turbidity, and reduction of chlorine and abnormality in PH.

C. Emergency rescue operation

(A).Process emergency respond operation based on the division of labor.
(B) Track down pollution source, pollution material, pollution situation and polluted area, and then gather water samples for examination.
(C) Shut down water purification operation at area with polluted water, and stop polluted raw water to enter water supply area.
(D) Process water drainage operation at the polluted zone.
(E) Clean out the polluted equipment or repair the damaged equipments.
(F) If necessary, implement district water rationing and water transfer operation.
(G) Announce water suspension, and notify the pollution situation, water suspension range, water recovery period through electronic or print media to users.
(H).Make visit to the polluted water supply area to appreciate water pollution range and understand the damaged and scope of effect. If anyone gets sick from situation, the person should be assisted to medical institution. Then, consolation and deposition should be made and the superior should be notified to request for assistance.

5) Emergency power failure operation:

A. Power failure protection operation:

(A). In time of power failure due to malfunction of power generator and power transmission caused by natural disasters, self powered generator should be used for emergency procedure.

(B) High-voltage and power distribution equipment along with self powered generator at water purification facility and compression station should be installed and maintained periodically, and they should be inspected periodically, while the deteriorated components should be replaced and repaired immediately.

B. Emergency rescue operation

(A).Process emergency respond operation based on division of labor.

(B).When outside power source is cut off, contact the power company for maintenance. Then, operate the reserve generator after understanding the cause of power failure and estimation of recovery time for power. Ready with self powered generator.

(C) Examine the recovered machines one by one and operate them.

(E) If internal high voltage and electricity supply equipment malfunctions, they should be repaired immediately or manufactures should be asked for,

(F) If controlling equipment malfunctions, use manual operation or activate backup machines.

6) Emergency leakage operation for sodium hypochlorite

Sodium hypochlorite is a strong-base erosive chemical substance. If the substance leaks, it will affect underground water, downstream river, or other water sources, and cause death of large quantity of fisheries and crops. It might also cause damage to people or cause secondary disasters.

A. Ordinary prevention operation:

(A). Inspect the storage tanks, flocculent feeding machine, and pipelines at monthly basis. PE hoses should be replaced each year to increase flocculent feeding machine's shock resistance capability.

(B).Inspect the appearance of the storage tank and flocculent feeding machine or the downstream sewage and check for leakage.

(C).The air circulation should be well kept in flocculent feeding machine room.

B. Emergency rescue operation

(A) If cracks are found at storage tanks or on the pipelines, the chemicals should be removed immediately to backup storage. Then the manufacture should be notified and scheduled for repairs.

(B).If leakage occurs, the downstream aquaculture, Water conservation irrigation, and other related units should be inform to perform blockage procedure and stop the pollution to prevent the spread of damage.

(C).If the repair or rescue becomes inefficient as it is unable to add chemical, dripping method should be used above the pool to ensure normal water quality.

To prevent flooding and effectively perform disaster rescue operation as well as increasing efficiency of operation time. "Disaster Prevention and Protection Act" is hereby based on to formulate every level of disaster prevention and rescue system. Once, complete and effective report and warning capability is established, it can properly handle the problems such as power failure, pipeline rupture, insufficient water supply, and sodium hypochlorite leakage caused by earthquake as well as the cleaning up and reconstruction operation so as to reduce damage to the minimum and improve the quality of water supply for general public. In order to take hold on with the golden timing, SCADA system and GIS equipment should be reinforced to realize notification mechanism to allow personnel to receive first hand information as they conducting efficient settlement with rescue operation and decrease the damage to minimum. Also, with sufficient supply of reserved equipment and rescue operation personnel, it will increase operation efficiency and decrease the rescue time. Investigation conducted on the scene will be used to evaluate the cause and necessities for recovery as rescue recovery operation will be conducted with priorities. Then, the group of personnel is divided into different divisions to perform rescue operations. The investigation record and evidences will be stored, and the information is presented and then redistributed after the cause of the incident is discussed. Education and trainings should be held every year to understand the possibilities and the proficiencies of each plan, and reviews and modifications are made for each operation plan. Planning and execution are both important. The simulation and planning for each emergency operation should be done regularly during normal days, while simulation exercises should be carried out and recorded periodically. The plans should be modified and changed based on the results gained from simulations for perfection.

4. ESTABLISHMENT AND APPLICATION OF CURRENT INFORMATION TECHNOLOGY

In the past, policy maker would need to go through personnel from leves l to transfer information related to the production, information delivery, wellbeing of facilities, and configurations of the pipelines. However, this delivery method is inefficient and might cause information to be incomplete. Today, we require speed to process information, and with the advancement of communication technology, eight branch management office has also integrated SCADA, GIS, Computerized Maintenance Management System, CMMS, Global Positioning System, GPS, Personal Digital Assistant, PDA, Mobile Computer, and Mobile Data Virtual Private Network, MDVPN into the communication system. They can help collect and effectively process large amount of information, which can increase the process and increase efficiency to reduce the need for manual operation and examination. The following is the implementation and application of the information technology in the eighth branch management division.

1) SCADA

A. scenario of establishment

In 2002, water purification plants and pressurizing plants in the service of jurisdictions, excluding Toucheng system, have been integrated and dispersed into 5 SCADA service sites. In 2007, they were integrated once more, including Toucheng system, and they are reduced into two service sites. The software and hardware were improved, and the transmission cables were replaced from low speed PSTN modem to symmetric Digital Subscriber Line, ADSL and Fiber to the Building, FTT to construct a Virtual Private Network, VPN to increase data transmission efficiency. To provide the policy makers and supporting staffs about manufacturing and information transmission, Web-SCADA is installed in 2008, and it has increased with management analysis interface and it uses GPRS and related mobile network technology to access to network traffic information. It is expected in 2009 mobile GPRS&GPS pressure monitoring equipment and its management interface will be completed to replace manual observation. Then, the information will be updated automatically into the database and increase the run-time report ability to the network traffic.

B. Application

SCADA provides monitoring and recording capabilities on water pump, water gauge, water meter, hydraulic pressure gauge, and water quality testing equipments at water purification plant and pressurizing station. It also provides controlling and recording function on electric valve, water meter, hydraulic pressure gauge, and water quality testing equipments at outer water network stations. PSTN modem connection and GPRS mobile network connections can be used at outer station and small stations. Graphic interfaces are used for equipment and controllers at the station for easy access to information. Then, VPN broadband network allows us to transmit surveillance information and provide centralized control to different areas as it is found with complete trends chart, alarm, and recording function. For outer station and smaller stations, the information is centralized with control station – SCADA, and it has system backup mechanism that uses dual main control computer and dual broadband network. It is used to monitor pressure and flow rate, and can perform pressure invert control to allow the water pump to be controlled with frequency conversion. The pumps can be set to increase or decrease its speeds and reduce pressure surge to prevent pipeline impact. The related images of SCADA are shown on chart 4. Web-SCADA is to place monitored information of SCADA with OPC technology and convert it into web-based human-computer interface, and it combines with supplementary management policy analysis for sharing the information. The related operation image is shown on chart 5, including the leakage inspection interface to provide warning analysis and help decide the operation area needed for leakage inspection. The information is integrated for the management for cross-analysis easily, which can increase decision making and disaster management. It is expected that mobile GPRS&GPS pressure monitor device and their management interface will be completed, and these equipment can change their monitoring position based on mission needs. The information from quick positioning and monitoring can be sent back to the database through GPRS&GPS, and be integrated with GIS. Especially when communication network is cut off during disasters, it can satisfy the needs for emergency response, showing great mobility efficiency.

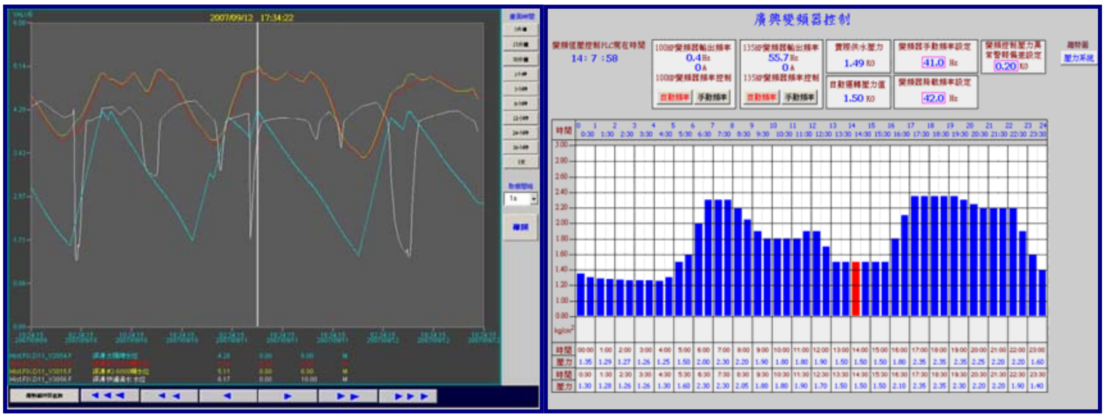


Chart 4Current SCADA processing chart at Eighth management area

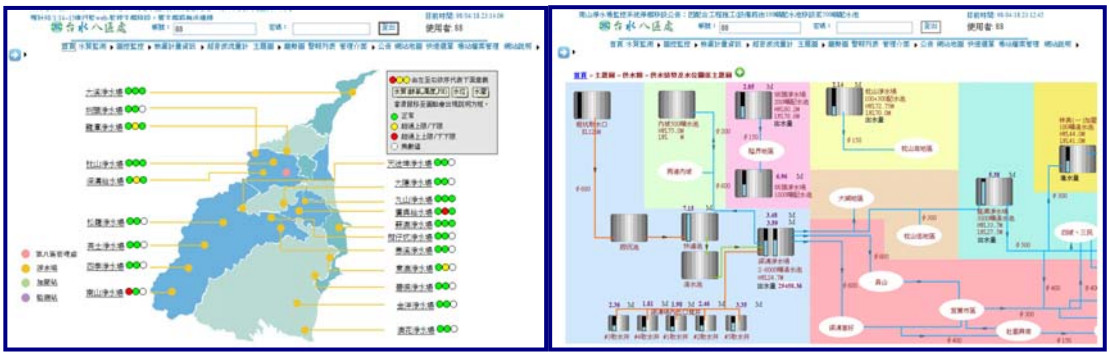


Chart 5(A) Current Web-SCADA processed image at Eighth management area

Chart 5(B) Current Web-SCADA processes image at Eighth management area

2) GIS

A. Scenario of establishment

In 2006, Yilan County Government provided the basic terrain map for the construction of GIS, and the scope of implementation area include the entire full water providing zone at Yilan area. In 2006, digitalization and the update of data attribute link were completed. In order to increase the accuracy of the map, it has included the map position and attribute information. In 2007, corrigendum operation for GIS was done piece by piece, and the locations of the equipment was placed on the map, including pipelines, valve systems, fire hydrants, and hand holes with the accuracy rate reaching 99%. The accuracy rate for pipeline's layouts reaches 82%. The information with the positions of valve systems, fire hydrants, and hand holes are still being updated, but the position layout of water meter has reached 100%. To use the system and promote work efficiency, in 2007, the employees were trained through on-job training to use the GIS system, and field test was given, and 117 employees participated, 80% of the total employees. It was required that operations and procedures should be familiarized. At the moment, GIS map is still being updated, and it includes the update of terrain map, completed construction, and leakage repairing construction, which includes error correction for the updated map.

Chart 6 Webpage inquiry system of GIS

B. Application

(A) PDA and GPS assistance for valve and hydrant inspection and leakage positioning management system.

When valve and hydrant facilities patrol personnel is performing inspection, their PDA equipped with GIS map can be used to inquire and register the inspected information related to the facilities to replace current recording card system. For example, it can help locate the position of valve systems, fire hydrants, and manholes, update the maintenance information, provide instructions on the positions and mark the leaking target, record the facility's situation, and take pictures and report maintenance related operation. It can also upload the information of inspection result to be directly to a PC and print them out through a printer after the operation is completed. Then, the information can be updated, while follow-up operation such as maintenance control will be initiated. Through PDA, the maintenance information is completely feeding back to the area's GIS after synchronizing update module and management module so as to maintain the consistency of data in the main system with data in GIS. The PDA operation is shown on chart 7. When a special situation occurs at a facility, GPS can be used to position the current layer and take picture to record the situation and feedback of live event, which can speed up follow-up maintenance and increase the efficiency of construction.

A. Proceed for checking the maintenance record for the valve system.

B. Confirm the data to be correct, and check to select for examine the damage or abnormality on the valve.

C. Current picture can provide confirmation on the location of the equipment. New picture can be updated if the location has changed.

D. If the valve is buried.

E. Display the triangular measurement for reference.

F. Data from the triangular measurement.

G. Surveillance result can be examine after the data is transferred from PDA to PC

H. Export the form to provide for maintenance tracking.

	ID	SYSTEM_ID	UNIFIC_ID	MAINT_DATE	MAINT_TIME	MAINT_ITEM	AREA
1	4	1766	0862000329173800400004	20080918	17:41:22	檢查正常	000003
2	5	1766	0862000329173800400004	20080919	10:57:11	檢查正常	000003
3	8	1766	0862000329173800400004	20080918	17:41:22	檢查正常	000003
4	9	1766	0862000329173800400004	20080919	10:57:11	檢查正常	000003
5	10	1766	0862000329173800400004	20080918	17:41:22	檢查正常	000003
6	11	1766	0862000329173800400004	20080919	10:57:11	檢查正常	000003
7	12	1766	0862000329173800400004	20080918	17:41:22	檢查正常	000003
8	13	1766	0862000329173800400004	20080919	10:57:11	檢查正常	000003
9	14	1766	0862000329173800400004	20080918	17:41:22	檢查正常	000003
10	15	1766	0862000329173800400004	20080919	10:57:11	檢查正常	000003
11	16	1766	0862000329173800400004	20080918	17:41:22	檢查正常	000003
12	17	1766	0862000329173800400004	20080919	10:57:11	檢查正常	000003
13	19	8527	0861003322973780400010	20081021	13:25:15	埋沒	000006
14	20	8527	0861003322973780400010	20081021	13:25:15	積泥	000006

Chart 7 operation application of PDA auxiliary valve

(B) District Meter Area management and planning system.

Construct district meter area (DMA) and divide the areas by meter reading areas, which can improve the problem with range of meter reading area that cannot coordinate with meter reading time and cannot integrate the information in DMA using paper works., for examples: the basic information of pipeline layouts (pipelines, valves and hydrants), the supply water in the DMA water sales in the DMA, the overlay analysis and application of GIS and establish DMA water supply management system. Then, long term observation databank can be established through reintegration of DMAs, meter reading zones, and water supply-and-sale analysis as it can perform long term recordings to each medium and small areas as well as examine the history of water sales in each area completely and efficiently so as to effectively, reduce water leakage and improve overall water sales.

(C) The integration of GIS and SCADA

It brings the images of Web-SCADA monitor station and other facilities to be displayed at GIS map’s station icon, and will connect to the monitor screen upon clicking, and authorized user can use it to control advance operations. In the future, important operation information for water purification plant or important parts of the network such as the water pump, electricity, flow rate, water level, water quality, generator status, and pressure, will be displayed by geographical location. Also, Diagnostics LED display is used to present current situation, such as red light: danger, yellow light: alert, green light: normal, and the information will be cross examined with characteristics of the pipelines. It can connect and open GIS platform, and will perform screening calculation with the pipelines’ characteristics to provide user with more convenient operating experiences.

3) CMMS

It digitalizes equipment maintenance operation for each water plant and implements the one card per one machine operation. The information stored at the equipment card includes basic information, user records, usual manual, standard operation procedure, maintenance record, repair record, and evaluation of effectiveness. The warranty work order procedure is used to track operation progress, while necessary backup components will be stored in stock so that it can provide managers with conditional query and printing function. Besides, it makes use of database to conduct classifications searches such as people, equipments, area, jobs, and funding as well as working hour statistics, equipment damage frequency, and maintenance costs, which is used to analyze the weak points at the equipment, effectiveness of material components or performance warranty. Furthermore, it also provides information of equipment maintenance status and spare parts management. When a disaster occurs, the information related to the equipment specifications, user’s manual, SOP, manufacture, Maintenance Company and inventory of spare parts can be access and viewed to increase the processing of situation. The system is nearly completed of its establishment and it is expected to be introduced by 2009. In the future, it is considered to integrate functions of RFID, PDA and MDVPN to increase work efficiency.

Chart 8 operation image of CMMS

4) Mobile computer

The development of mobile computer is also a noteworthy event. Currently, there are Mobile Internet Device, MID, Ultra-Mobile PC, UMPC, and Small Notebook PC that are useful to connect to internet or perform information operation outside of office. They are small and handy, but with development from different fields, other small sizes of equipment such as Touch panel, 3.5G mobile network, browser, and GPS were developed. It is expected that in 2009 3.5G MDVPN's small notebook PC will be purchased, and engineering staffs or maintenance personnel can access to GIS or Web-SCADA anywhere outside to inquire pipelines maps and monitor the stations. Then they are able to expend and partner with RFID reader or barcode reader and combine with CMMS with on maintenance operation.

5. APPLYING INFORMATION TECHNOLOGY ONTO DISASTER RESPONSE PLAN OF EARTHQUAKE AND FUTURE PLANNING

A modern water supply and purification plant management should make full use of information technology so that, during time of emergency the response center can obtain full grasp of the situation and the progress to perform accurate resource allocation to increase operation efficiency. The structure of information management is shown on chart 9. Each information system can transmit information between one another or perform data processing, and it can provide integrated graphical information to policy makers to make the best decision. At the moment, the eighth branch is promoting to establish and integration of SCADA, GIS, and CMMS. Besides the application of current information technology described in the previous chapter, the following is an illustration of the integration and application of information technology used during earthquake disaster.

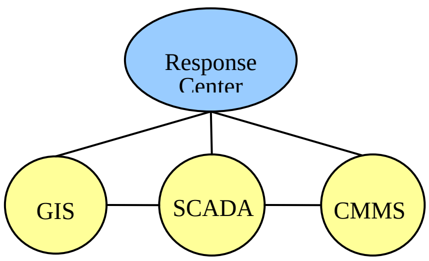


Chart 9 Application of communication management structure for disaster response

1) distribution reservoir display indicators

By using the information collected from SCADA, an independent frame is established showing the distribution of reservoir's water level in previous six hours with each data line showing every half hour. The frame marks the upper limit, lower limit, uppermost limit, and the lowest limit of water level, and also the information of height of the overflow pipe and reservoir, elevation of water tank, and the reservoir's surface area. Then, overall information is calculated into (1) inefficient stock (2) supporting time limit. These calculations means when the entry end of the distribution reservoir stops sending in water, the distribution reservoir can buffer the amount and time of water yield. The amount of time needed is calculated from the water yield from previous hour (or half an hour, or two hours, time can be randomly picked). By using water level information from each pool, the water filling period can be estimated, and the risk resistance capability evaluated, while the information used as a reference to consider the order using reserve materials.

2) chart display of water level relationship

It uses SCADA and puts water levels in each stations, water quantity, and water pressure as combined with pipeline information from GIS, which includes water quantity in water consuming area and the pressure to be displayed by high level allocation and real-time information. It is used as reference for emergency water distribution. The personnel at the response center can use changes between the water quality, water pressure, and water level to determine the range of damage for water facility.

3) condition display of station operation

Use GIS geographic distribution to display the conditions of water purification plant and pressurize station. User can select the conditions on water pump, electricity, water quantity, water level, water quality, and the condition of the generator, and the indication display can be used as indications of conditions for the systems described previously, while diagnostics are displayed by LED lights. The LEDs are presented as: red light: danger, yellow light: warning, green light: normal. Detail information can be accessed by clicking on the lights, and they are used for supervision and management purposes during emergency situation.

4) display of water supply

With use of the information collected by SCADA, one can establish Water supply status map, and can provide the information about water quantity, water reservation quantity, pipeline pressure, gasoline left in the generator and running hours, the updated meter reading information, water consumer population, water consuming households, and the distribution of industries and businesses in real time. During emergency period, the information can be used to grasp water distribution situation. It can also combine with water level relation chart, and provide as a supporting tool for managers to arrange support of water supply. Besides, it can create many scenario scripts simulating possible abnormalities as they are stored in the database to be used as procedure outlines to handle emergency responses, and can guide the operators to perform immediate response.

5) generator display

GIS can be used to show the information of performances of the electric generator. During an emergency period, the information at the situation the generator is very important. The display can calculate the generating capacity and fuel consumption curve, and then preparation time can be calculated using the real-time fuel inventory. It allows the user to get hold onto the preparation status with the electric generation, so that gasoline supply operation can be arranged properly.

6) security information display

It is found with the integration of surveillance images from current water plant's security video system, the CCTV, and by adding the surveillance image from the management center and the business lobby, fire information, oil storage in the water purification plant, detection of chemical leakage, intrusion detection, smoke detection in the generator room, and temperature detection. It will sound out alarm during abnormality to provide full cover safety maintenance. The video images can also be used to obtain real-time situations at disaster areas.

7) station communication networking

A traditional control equipment PLC or RTU uses serial data communication or SCADA connection. However, when it exploits serial data communication on decentralized management, because its bandwidth is too narrow, it would create the problems of inefficient data transmission quantity and slow examinations through telecommunication lines. Therefore, telecommunication network and mobile network should be updated. Then, through VPN network it can connect with decentralized station and control equipment, and it can provide speedy connection to conduct distance equipment maintenance and collect more and delicate information to use for back-end control or analysis. General speaking, maintenance of mobile network is faster than using telecommunication network, especially when telecommunication network is damaged by earthquake. If this happens, backup support network and mobile network can be used to make SCADA continue to provide service, so it ensures a stabilized, safe, and high quality information transmission. It can also construct communication network management interface to track abnormalities in the communication network. During emergency times, for coordination with the establishment of county government's response center, the stationed personnel can use mobile network and connect to the area management office to inquire water purification's operation information, water supply zones, the information on the preparation of generators, response plans, personnel on duty information, and disaster situation report. It can also be used to report information in order to reach the goal of rapid response process.

8) widespread installment of pipeline network for tracking analysis and control device

When performing traditional pipeline network online supervision such as pressure, water quality, and water flow rate, it would need to operate with communications box and a base, and apply for electric power and telecommunications equipment schedule. Therefore, it is difficult to master, and it could not meet the needs for emergencies and mobility. The eighth division management center has brought forth the solution plan that does not require electric power, telecommunication equipment schedule for it is equipped with GPS system. The mobile supervision equipment uses low-power product, and it uses batteries to provide electricity. Then, through mobile networking, it can return surveillance data, and position the location immediately by GPS receiver. Then, data will be displayed directly on GIS platform as it can accomplish surveillance service using geographical mapping. It is expected in 2009 the mobile GPRS&GPS pressure monitor equipment and its management interface will be completed, and in the future it will also meet the information need to obtain data for water quality and water quantity. Through the installment of wide-spread long-term pipeline network for tracking analysis and control device, it can monitor the flow rate and pressure for an extensive period of time and control the electric valve, and it can also establish display board. For the pressure section, GIS can present pressure energy gradient map. When abnormality pressure occurs in a specific area, it can be located and the situation understood through geographic layout map. Then, it can analyze pipeline network problem by collocating with water flow situation. For electric valve, first electric valve will be installed in the important nodes, and it will combine with SCADA to set up various control modes to perform water quantity and pressure management. Afterwards, when ruptured pipeline or large amount of water outlet due to fire extinguishing is discovered, other water supply abnormalities can all be handled through remotely control of the electric valve. The mobile pressure monitor device can be installed at the construction site or disaster area, and it can control pipeline network situate at the construction site especially when communication is cut off during a disaster. Moreover, when the pipeline is ruptured and fire has also occurred, the personnel at the response center can use SCADA to check the water pressure and water quantity in the fire area to locate the nearest area with the highest water pressure, and fire fighting unit can be notified to collect water. When serious water supply deficiency occurs, water supply stations installed at the communities can be equipped with GPRS mobile water level monitoring equipment as it can control the level of pools or water reserve tank from a distance to arrange the time for delivery of water supply.

9) earthquake disaster response strategy

Disaster response operation and sequence criterion are in relation to the deployment of emergency response resources. Therefore, strategies must be planned in advance. This

study proposes the sequence for repairing during disaster emergency should start from water purification plants, main water pipes, secondary water pipes, important areas, public facilities and hospitals, secondary water purification plants, secondary pipelines, and secondary water distribution pipelines. Then, repair process will be adjusted based on the situation of current material in-stock, supporting machines, and traffic situation. The repair condition is first to provide enough water, and then satisfy water pressure, and the last is to satisfy the demand for water quality. The direction to execute response strategy should be:
(A). Provide water needed for fire extinguishing.
(B). Provide emergency water for life support, water deployment, and insure water quality.
(C) The repair of water pipelines and deployment of resources.

Based on the water distribution regulation after Japan's Kobe earthquake, water need to maintain life support varies. Right after the quake, each person would need 3 liters to survive, and then it will rise to 100 liters on the 11^{th} day, and then 250 liters on the 22^{nd} day. These data is used to make policies for emergency water supply. The water supply truck will carry the water to the specific location, or water supply stations are set up in the communities which water will be provided periodically.

Establish the division of small water providing areas. After a disaster, water will be supplied to the unaffected areas. However, water supply for the areas under serious damage will be restricted and it can be controlled through emergency trip valve or remote control valve. After acknowledging disaster situation with confirmation after inspection, the water providing area will be adjusted and recovered one by one. Then, it is through the channels from the local government and disaster response headquarter, police station, fire station, and other disaster preventing organizations to maintain close communication with one another and conduct recovery construction and prevent secondary disaster. Reinforce the organization of consumer center, and pay visit to the local political unit or organization to explain the recovery situation. Try to obtain understanding and listen to the people's needs

10) estimate the range of earthquake disaster

After the water supply facility is seriously damaged, major water suspension will occur. Due to total water suspension, many patrol personnel are needed to be dispatched and perform surveillances. They will follow the water main pipe to repair and recover water supply, so that accurate correct damage areas can be discovered. Also, based on *Shih* Ban-Jwu team's research in 2004, the damage of life sustenance system has its relevance with the path of dislocation crust on PGD changing rate. Therefore, the information can be obtained from the weather bureau and the central earthquake center, and PGD changing rate can be used to estimate the damage route and range of inner pipelines, which can be used to plan the deployment of emergency resources.

11) earthquake disaster prevention education training

Take serious with earthquake disaster prevention education training. Disaster prevention drill should be hosted periodically to increase disaster prevention skills. At the time of disaster, through the periodic earthquake disaster prevention education training, people could provide quick and accurate responses. Also team cooperation and disaster prevention skills can be promoted by taking part in disaster prevention drills hosted by various organizations.

By combining resources mentioned above, the eighth branch has proposed an earthquake disaster response plan, which is shown in the flow chart at chart 10. When an earthquake occurs, emergency response center is established by the eighth branch based on the regulation. The personnel in the center will integrate SCADA, GIS and CMMS communication system to observe data changes from major facilities' indicators such as water quantity, water pressure, water level and water quality. The information obtained is to be used to estimate the situation and range of possible damage that could happen to the facility, and then the patrol personnel will be sent to the area with possible damage. The graphical information of pipeline, facility's basic information patrol, and inspection form can be downloaded and stored by GIS in PDA and carried by patrol personnel. The PDA also has photo option that can be used to check whether the pool and related structures are damaged, whether the electrical control equipment has affected water supply, whether the pipeline network has ruptured or mechanical malfunction, and whether the water source is polluted. In the future PDA might become a mobile computer equipped with 3.5G network. Then it can connect with SCADA, GIS and CMMS in real-time. Information can be accessed based on the practical needs, and information at the site can be transferred back to the response center. The personnel at the center can analyze the information to help policy makers to provide a response policy quickly. After the disaster is confirmed, patrol personnel will be notified so that they can perform emergency processing such as shut down the valve system to decrease the affective rage, and at the same time the machine will be switched to manual operation of water distribution mode, such as deploying water for nearby purification plants. Depending on range of disaster, backup support team will also be initiated to conduct restoration operation. With application of information technology and innovative technology to water company, it can help to reduce manpower needs and reduce response time, allowing the personnel to operate with ease and increase emergency response operation efficiency.

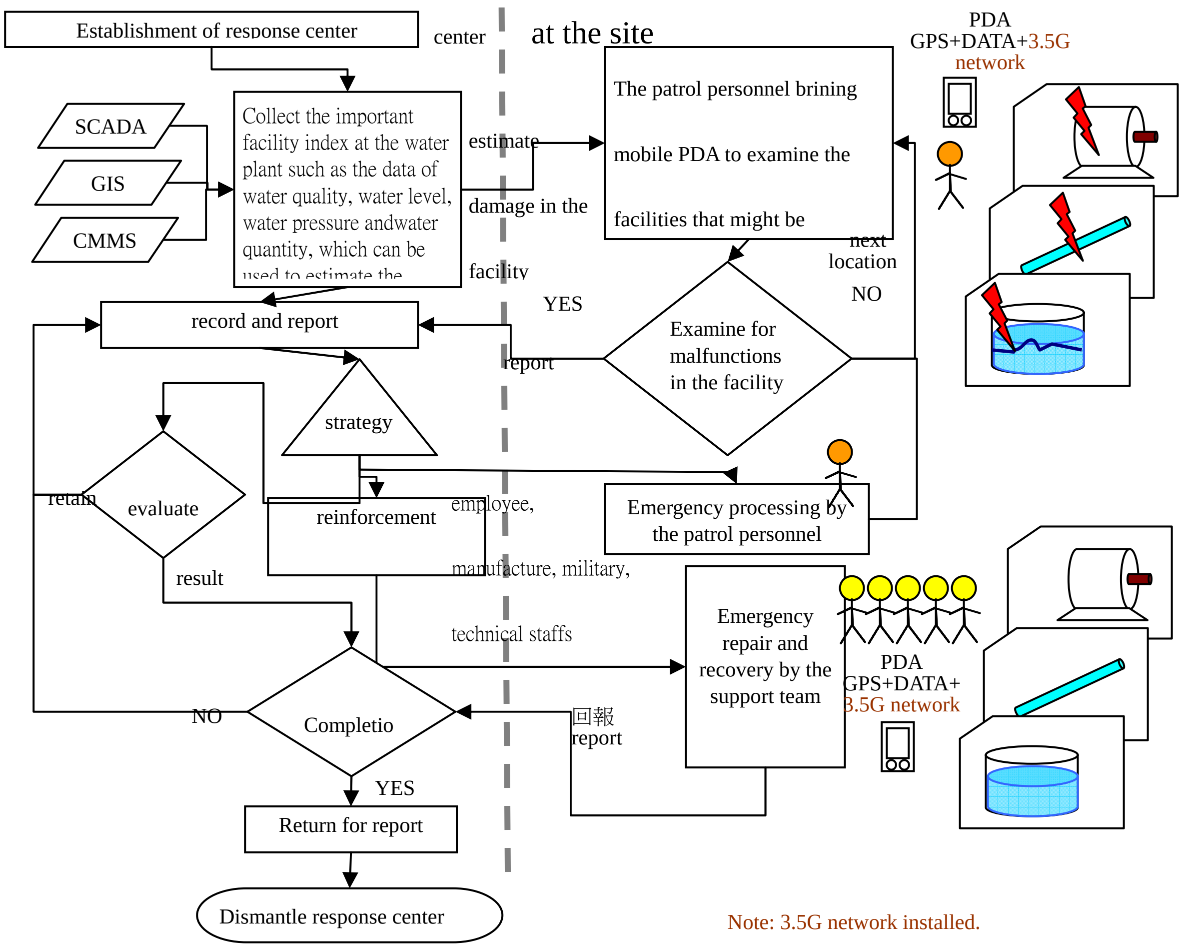


Chart 10 Earthquake disaster response procedures of the he eighth branch management center

REFERENCES

1. Central weather bureau web site www.cwb.gov.tw, 2009.
2. Construction and Planning Agency, Ministry of the Interior, Buildings' quake resistance design area division in Taiwan.
3. National Disaster Prevention and Response Committee, the Executive Yuan, Disaster Prevention and Rescue Basic Plan 2007.
4. National Disaster Prevention and Response Committee, the Executive Yuan, earthquake disaster prevention and rescue plan. 2009.
5. Taiwan Water Corporation, disaster emergency response team outline. 2009.
6. Taiwan water corporation eighth branch management center, emergency respond plan, 2009.
7. Shi, Bang-zhu, Luo, Jun-xiong, Tsai, Jin-song, Chen, Wei-yao, Tian, Yao-zhang, Feng, Zhong, Zhou, Lide, and Zhang Zhe-hao, Lifeline system earthquake criticality analysis and integration research, national science council project report NSC 90-2625-Z-027-002, 2004.
8. Zhang, Tangzheng, "The estimation and countermeasures of shock damage to the water pipelines – Using Chiayi city as example," National Taipei University of Technology institute of civil and disaster prevention Dissertation for Master's Degree, 2004.

Continuous Earthquake Preparedness – Strategic Investment of Resources for Optimal Earthquake Response

Walter J. Bishop and Stephen J. Welch
Contra Costa Water District

INTRODUCTION

The Contra Costa Water District (CCWD, District) provides untreated and treated water service to over 550,000 customers in Contra Costa County, California. Its system draws water from the Sacramento-San Joaquin Delta and consists of a 70-year old, 70-kilometer concrete-lined open canal which serves three water treatment plants to distribute water through over 1,400 km of water pipelines using 36 pump stations and 41 reservoirs. CCWD's customers are municipalities, industry, agriculture and urban residential.

In 1994, following the Loma Prieta and Northridge earthquakes, CCWD undertook a comprehensive seismic assessment of its water conveyance, treatment and distribution systems. The study focus was to identify strategic improvements throughout the CCWD system to minimize water service interruption after a maximum credible earthquake. The study identified over $200 million (2009 U.S. dollars) of improvements required in the system, including improvements to existing pumping and piping, as well as construction of additional pumping and conveyance (both piping and canal) improvements.

Since that initial effort, CCWD has made significant additional investment in both operational and emergency response reliability in its system. It has invested over $50 million (2009 U.S. dollars) of improvements in existing pump stations, storage, pipelines and valving to meet the objective of reliable emergency operations with a major focus on post-earthquake response.

Through these investments, CCWD places a strong focus on developing a sustainable water system under all conditions, including post-earthquake operations. CCWD ensures its investments balance improvements to its capital assets with training and preparedness of its staff and management for operations under emergency conditions.

This paper will present CCWD's steps to meet these objectives including its investments in major upgrades of its water infrastructure over the last fifteen years since the Loma Prieta earthquake; its ongoing evaluation and planning to ensure its new and existing facilities will perform as needed following a major Bay Area earthquake; and its ongoing program to ensure its managers and staff are trained and prepared to perform under the conditions anticipated following a major earthquake event, or other similar disaster or emergency.

OUTLINE OF CONTRA COSTA WATER DISTRICT

CCWD was established in 1936 to provide water to the central and northeastern regions of Contra Costa County, California. CCWD headquarters is in Concord, California, approximately 56 kilometers east of San Francisco, located in Central Contra Costa County. (See Figure 1)

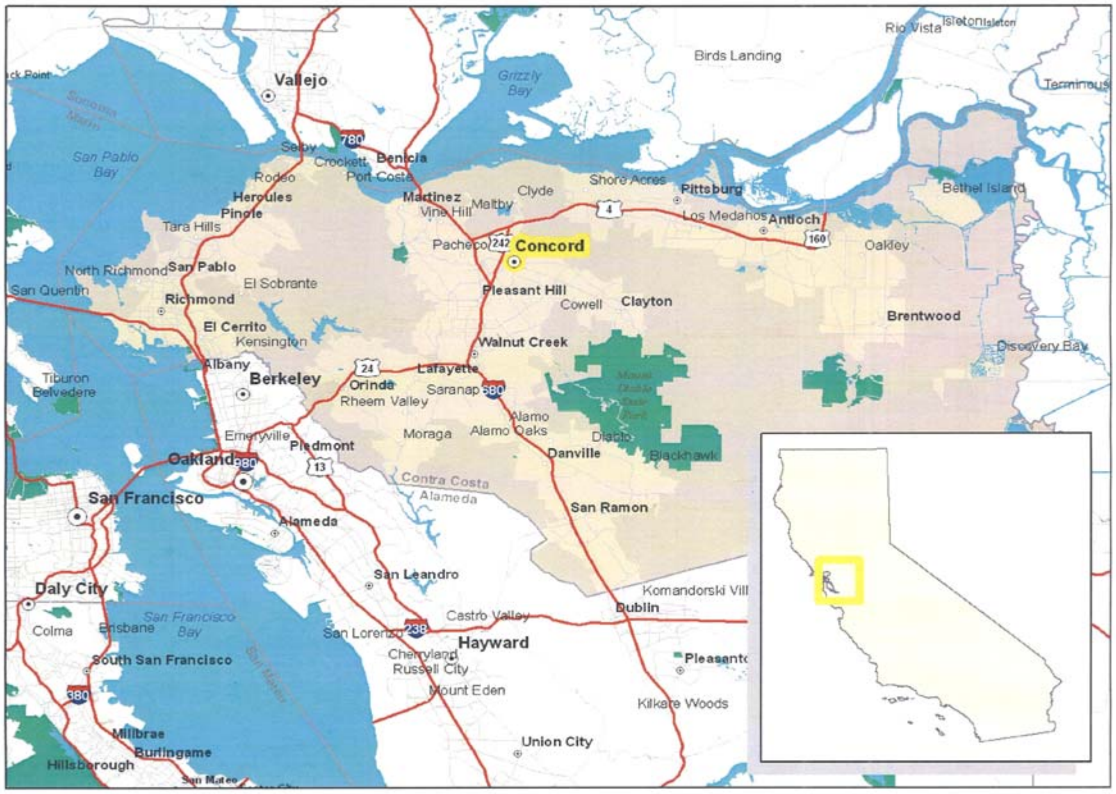


Figure 1 – Vicinity Map

CCWD draws its water from the Sacramento-San Joaquin Delta under a contract with the U.S. Central Valley Project (CVP) through the United States Bureau of Reclamation. As part of the CVP, the 80-kilometer Contra Costa Canal (70-kilometer concrete-lined canal; 10-kilometer unlined canal) was built in 1937. By means of the canal, water is diverted from Rock Slough (13-kilometers east of Antioch, California) through the 10-kilometer unlined channel into a 70-kilometer concrete-lined canal. Four pump stations lift water 38-meters above sea level to the canal's Antioch summit, after which gravity flows the water to its terminus in Martinez. The canal runs through Oakley, Antioch, Pittsburg, Concord, Walnut Creek, Pleasant Hill, Pacheco, ending at a terminal reservoir in Martinez, California (see Figure 2).

Contra Costa Water District
Service Area Map

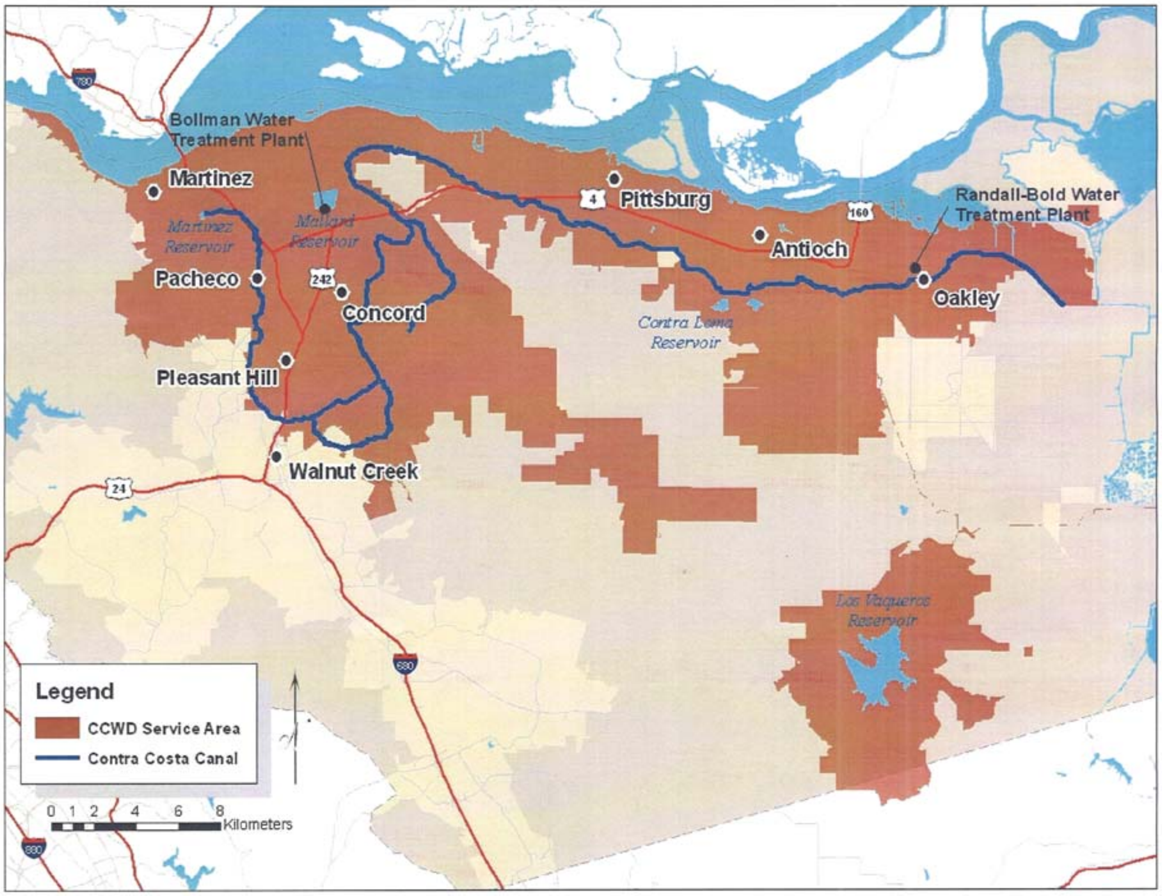


Figure 2 – CCWD Service Area

CCWD has four untreated water storage reservoirs: Martinez, Contra Loma, Mallard, and Los Vaqueros, totaling a storage capacity of 1,233,500-litres (103,070 acre-feet). It also has three treatment plants; the Ralph D. Bollman Water Treatment in Concord, with a capacity of over 300 million litres per day; the Randall-Bold Water Treatment Plant in Oakley, with a capacity of 200 million litres per day; and the City of Brentwood/CCWD Water Treatment Plant in Oakley, with a capacity of 60 million litres per day. Roughly half of CCWD customers receive treated water directly from CCWD, the remaining from five local agencies who treat and distribute CCWD water.

Today, Contra Costa Water District serves a population of over 550,000 and includes treated water distribution facilities with 36 pump stations, 41 storage reservoirs, and 1,252-kilometers of pipelines. CCWD is managed by a General Manager appointed and governed by a five-member, publicly elected, Board of Directors. CCWD's 325 employees serve customers that include the municipalities of Antioch, Pittsburg, Brentwood and Martinez, commercial and industrial companies, agricultural entities, and the residential communities of Clayton, Concord, Pleasant Hill, and Walnut Creek

The greatest challenge CCWD faces today is continuing to provide high-quality water with an ever-increasing customer demand in an environment with an extremely limited water supply.

Such increasing demands are continuously increasing the consequences of damage from an earthquake or other disaster, further increasing the importance of CCWD's duty to provide reliable water. CCWD is committed to ensuring adequate water resources, high water quality, and reliability for the present and future, particularly in times of emergency.

EARTHQUAKE PLANNING FOR SYSTEM STRATEGIC CAPITAL INVESTMENTS

Following the Loma Prieta and Northridge earthquakes, in 1994 CCWD undertook a comprehensive seismic assessment of its water conveyance, treatment and distribution systems. The study focused on strategic improvements throughout the CCWD system, and identified over $200 million (2009 U.S. dollars) of improvements to existing pumping and piping, storage and power systems. CCWD's resulting Seismic Reliability Improvements Program (SRIP) was created to implement these improvements.

The purpose of the SRIP was to identify a combination of capital and operational improvements that efficiently allowed CCWD to provide reliable water service. The SRIP initially defined reliability criteria and seismic design criteria. This first step included defining the study area (CCWD existing and planned untreated and treated water service boundaries, see Figure 2); the planning period (out to the year 2020); the specific facilities to be studied; and the analysis stress events by which to model (Concord fault, M6.5; Great Valley fault M7.0). The criteria was intended to be comprehensive of both expected deficiencies of the normal operating system, and emergency based deficiencies resulting from the stress event.

Since 1994 the District has implemented the complete SRIP. The implementation was completed through two key efforts. The first was construction of the Multi-Purpose Pipeline (MPP), a 45-kilometer, 122-centimeter backbone transmission pipeline designed to ensure a reliable post-earthquake source of either treated or untreated water. The second was construction of seismic improvements through CCWD's existing Capital Improvement Program. From 1994 to the present CCWD had planned out capital improvements at various sites throughout its treatment and distribution system. These improvements were not necessarily entirely for earthquake preparedness, but in the interest of efficiency, interrupting the site and community just once, and ensuring upgrades were comprehensive, CCWD implemented the seismic improvements (as identified in the SRIP studies) through already planned capital improvements projects. To ensure the upgrades were implemented with a criteria based on overall need, improvements were prioritized based upon both operational need and seismic importance. As a result, though many sites were already planned for improvement over the 10-year horizon, some were re-prioritized to be completed earlier, while other sites were added as a result of no previous planned improvements during the implementation horizon. (Table I outlines the sites and years of implementation.) Additionally, as the improvements were being implemented within the view of an overall improvement to the facility (both seismic and operational), designs oftentimes solved both a seismic as well as operational deficiencies. (For example, an unanchored interior-to-tank inlet/outlet pipeline was not simply anchored, but replaced with a new multi-directional, diffused water distribution pipeline to address both seismic and water quality issues simultaneously).

The improvements implemented by CCWD over the last 10 years have been predominantly applied to CCWD's treated water system. About 90-percent of the efforts have been treated water. Table I provides an overview of the improvements made.

TABLE I – SEISMIC PROJECTS SUMMARY (EXCLUDES MPP)

Site	Description	Year Implemented	Project Bid	Seismic Cost (Construction)
TREATED WATER				
Nob Hill Tank	Coating, structural, operational Rehab	2000	$250,000	$35,000
Divide Tanks	Coating, structural, operational Rehab	2001	$307,000	$25,000
Clayton Valley Tanks	Coating, structural, operational Rehab	2002	$709,000	$135,000
Port Costa Tank	Tank Replacement	2002	$500,000	$150,000
Elderwood Tank	Coating, structural, operational Rehab	2005	$1,505,000	$350,000
Bailey Pump Station	Pump, electrical, structural, coating Rehab	2000	$755,000	$45,000
Pine Hollow Pump Station	Pump, electrical, structural, coating Rehab	2004	$504,000	$55,000
Clayton Valley Pump Station	Pump, electrical, structural, coating Rehab	2002	$954,000	$65,000
Elderwood Pump Station	Pump, electrical, structural, coating Rehab	2001	$775,000	$85,000
Paso Nogal Pump Station	Pump, electrical, structural, coating Rehab	2004	$659,000	$72,000
Lime Ridge Pump Station	Pump, electrical, structural, coating Rehab	2005	$1,942,000	$285,000
TW Generators/Seismic Valves	Backup generators to key pumping facilities	2004-2005	$1,675,000	$145,000 (seismic valves)
Seminary and Other Reservoirs	Coating, structural, operational Rehab	2007	$1,100,000	$150,000
Bollman 5kV	Electrical replacement, anchorage	2006-2007	$3,500,000	$100,000
San Miguel Pump Station	Pump, electrical, structural, coating, new building for seismic	2006	$2,600,000	$1,000,000
Bollman Building Improvements	Structural, architectural	2006	$220,000	$180,000
TW Reliability Fault Crossings	Pipeline upgrade	2007	$680,000	$680,000
Pittsburg Interties	Pipeline upgrade/intertie	2007	$340,000	$340,000
Bisso Building Generators	Electrical	2007	$700,000	$700,000
Seminary and Other Pump Stations	Pump, electrical, coatings, emergency power	2009	$400,000	$200,000
UNTREATED WATER				
Martinez Reservoir Outlet	Coating, structural rehab of outlet pipeline	2004	$501,000	$55,000
EOC Building Upgrades	Structural upgrade	2004	$394,000	$305,000
RW Reservoir Seismic	Seismic improvements	2001	$1,847,000	$1,847,000
RW Pumping Plants	Electrical, structural rehab	2003	$678,000	$55,000
Canal Check 2 and Bridges	Structural repairs/upgrade	2006	$400,000	$300,000
EBMUD Mokelumne to LVP Intertie	Pipeline Intertie	2007	$1,920,000	$1,920,000

Treated water

Core improvements to the treated water system consisted of facility structural retrofits (for example strengthening existing building walls, or roof to wall connections), backup power connections, addition of seismically activated valves, strengthening of pipeline supports, improved anchorage of system components, flexible connections at structure to pipeline connections, planning for interties with adjacent water systems, and external power generators at key pumping plants.

Table II provides a summary breakdown from the Table I list of sites for the seismic improvements made at each treated water facility. Except for the Lime Ridge Pump Station and EOC Building Rehabilitation projects which carried a significant share of cost for building strengthening, the seismic elements of the projects generally required less than 15-percent of the project funding. Through the use of sensible, basic, and easy to construct seismic engineering and design, CCWD has been able to prepare its system for the design earthquake for a reasonably low cost.

TABLE II – TREATED WATER PROJECTS SUMMARY – SEISMIC SCOPE

Site	Description of Seismic Scope Only
TREATED WATER	
Nob Hill Tank	Strengthened column, flex-tend to I/O, (advanced analysis verified tank)
Divide Tanks	Strengthened column, (advanced analysis verified tank adequate)
Clayton Valley Tanks	New tank holdowns, flexible overflow connections, strengthened rafter to roof to wall connections
Port Costa Tank	Tank Replacement - meet current code, flex-tend I/O
Elderwood Tank	Strengthened rafter to roof to wall connections, added rock anchors and foundation, flex-tend I/O
Bailey Pump Station	Strengthened pipe supports, flex connections at suction/discharge piping, equipment anchorage
Pine Hollow Pump Station	Strengthened pipe supports, flex connections at suction/discharge piping, equipment anchorage, anchored surge tank
Clayton Valley Pump Station	Strengthened pipe supports, flex connections at suction/discharge piping, equipment anchorage, advanced analysis of pump building, anchored surge tank
Elderwood Pump Station	Strengthened pipe supports, flex connections at suction/discharge piping, equipment anchorage, strengthened enclosure walls
Paso Nogal Pump Station	Strengthened pipe supports, flex connections at suction/discharge piping, equipment anchorage, strengthened enclosure walls
Lime Ridge Pump Station	Strengthened pipe supports, flex connections at suction/discharge piping, equipment anchorage, lowered roof to lower response (avoided new building cost), strengthened building walls
TW Generators/ Seismic Valves	See Table III
Bollman 5kV Electrical	Anchor newly installed electrical switchgear and conduits to current seismic code.
San Miguel Pump Station	Construct a new electrical building to replace the seismically deficient existing electrical building, brace piping and all equipment.
Bollman Building Improvements	Remove failing concrete architectural features to eliminate falling debris hazard.
TW Reliability Fault Crossings	Construct pipeline bypass facilities and valving at major transmission lines crossing the Concord fault.
Pittsburg Interties	Construct two emergency water supply interties between CCWD MPP and City of Pittsburg TW system.
Bisso Building Generator	Install an emergency power generator for the CCWD Engineering, Construction, Planning and O&M departments.
Seminary and Other Reservoirs	Anchor piping and make minor structural repairs to reinforce roof.

One of the more innovative approaches taken in implementing the seismic improvements was used for the Treated Water Generator and Seismic Valves (TWGSV) project. This project initially started as a project to add additional treated water storage throughout the distribution system. The project also provided for the addition of seismically activated shutoff valves at key, redundant treated water storage throughout the distribution system. The concept of the project was to add emergency response flow reliability by adding new tanks, and preserving existing tank storage volume through seismic valves. The estimated cost for this approach exceeded $10,000,000.

Instead of constructing more storage that mostly remained unused under normal operations, and could still simply drain away through significant pipeline ruptures following an earthquake, CCWD constructed more reliability into pumping capacity. Because the majority of the initial demand to the water system following an earthquake results from pipeline ruptures, not just fire fighting, additional storage is a one-use investment. As the major reason for pumping failures following an earthquake is from lost power, CCWD placed its focus on power reliability. CCWD performed optimization studies to identify the key pump stations, and the needed pumping capacity for emergency response demand conditions to develop a project that instead installed emergency power at nine key pumping stations throughout the system. Table III provides a summary of the power systems implemented. The result of the TWGSV project approach was a savings of over $8,000,000 to CCWD.

Table III lists the other minor effort of the TWGSV project, addition of reservoir outlet seismically activated valves. Such valves were required at four reservoir sites. The actuators were installed on existing valves where possible, and were only installed on reservoirs that provided redundant storage within the distribution system pressure zone to balance operational control and reliability.

TABLE III – GENERATOR SUMMARY BY SITE

TREATED WATER GENERATORS AND SEISMIC VALVES PROJECT **Project Summary**				
Generator Sites (Pump Stations)		Total Pumps at Station	Pumps Served by Generators	Generator Size
1	**BAILEY**	6	4	450 kW
2	**CLAYTON VALLEY**	5	3	300 kW
3	**GREGORY GARDENS**	3	1	150 kW
4	**LIME RIDGE**	4	2	400 kW
5	**PASO NOGAL**	3	2	100 kW
6	**PINE HOLLOW**	5	4	300 kW
7	**SAN MIGUEL**	5	3	300 kW
8	**EAGLE PEAK**	3	---	---
9	**SEMINARY**	3	---	---
Reservoir Sites (Seismic Valves)		**No./Size New Valves**	**No./Size New Operators**	**No./Size Seismic Valves**
1	**BAILEY**	2-24"	1-24"	1-24"
2	**CLAYTON VALLEY**	n/a	2-12"	2-12"
3	**LIME RIDGE**	1-36", 1-24"	1-36", 1-24"	1-36", 1-24"
4	**TAYLOR**	n/a	n/a	1-24", 1-16"
Generator Sites (Other)		**Total Pumps at Station**	**Pumps Served by Generators**	**Generator Size**
1	**BISSO LANE**	N/A	N/A	750 kW

Untreated water

Core improvements to the untreated water system consisted of canal facility improvements (gates and lining replacements), slope modifications (shallow the slopes), backup power to primary pumping, pipeline and fault crossing improvements, and an additional redundant untreated water pumping plant and piping.

Table IV provides a summary breakdown from the Table I list of sites for the seismic improvements made at each untreated water facility. CCWD's resulting cost effective solutions for earthquake preparedness ensure the objectives outlined above could be secured without rate impacts, and without limitations on CCWD service.

TABLE IV – UNTREATED WATER PROJECTS SUMMARY – SEISMIC SCOPE

Site	Description of Seismic Scope Only
UNTREATED WATER	
Martinez Reservoir Outlet	Installation of a 24-inch flex-tend to outlet
EOC Building Upgrades	Add shearwalls, roof to wall connections, additional roof ties, wall to foundation connections, added foundation
RW Reservoir Seismic	New outlet structure, new outlet piping, seismic actuated valves, new controls and valving
RW Pumping Plants	Strengthened walls, anchored equipment and piping
Mallard Slough PS	New pumping plant and piping – meet current code; not strictly seismic
Canal Check 2 and Bridges	Replace and strengthen seismically deficient bridges and canal structures.
EBMUD Mokelumne to LVP Intertie	Construct a new emergency (and limited water supply) intertie between CCWD and EBMUD systems.

CONTINUOUS IMPROVEMENT – SHARPENING THE SAW

CCWD has made significant past capital investments in ensuring its system is well prepared for a major earthquake. This does not mean CCWD's work is done. CCWD designs all new facilities with the latest earthquake preparedness provisions in mind, and is constantly evaluating its existing system, considering life cycle and changing conditions (including the latest earthquake knowledge) to ensure its system will perform effectively during an earthquake. As a result, CCWD includes ongoing capital investments through its Capital Improvement Program towards better seismic performance.

CCWD annually reviews its Capital Improvement Plan, consistent with its 10-year rate model and financial plan, to ensure it is continuing the District's strategy, as well as considering any changes in the environment (for example code or regulatory changes.) As was indicated above, CCWD has adjusted its planned projects for earthquake preparedness several times over the last fifteen years. New projects have been identified and implemented over that timeframe based on new information. For example, in 2004 the District undertook its TWGSV project to create a backbone system for water delivery. The project was initially identified as a distribution system improvement to add treated water storage in the system to address fire flow needs. However, as noted above, the optimized project provided far more benefits than emergency response alone. CCWD continuously evaluates the Capital Improvement Plan for opportunities as this to ensure its emergency preparedness and reliability can be strengthened along with other improvements as often as possible.

As an example of CCWD's continuous update of its emergency response needs, in 2008 the District identified the potential for several of its administration buildings, originally thought to be earthquake-safe, to fail under current seismic code requirements. CCWD conducted an evaluation of several of its key facilities, including its main administration building that houses its financial systems, its information systems, and its top management. The evaluation identified two collapse deficiencies, both at its primary exits, and these are being strengthened currently. The study also identified over $2 million of improvements necessary to ensure life-safety. These improvements are currently being planned into the annually updated Capital Improvement Plan for implementation in the next two years.

Another example of continuous evaluation and improvement is the recently identified Randall-Bold Water Treatment Plant (RBWTP) seismic improvements. The RBWTP completed construction in the early 1990's and was considered designed and constructed to the latest earthquake standards. CCWD wanted to confirm the adequacy of the plant, and in 2008 conducted a study of the plant's structural systems. The study identified nearly $1 million of structural improvements to strengthen existing buildings, electrical systems, tanks and equipment. The project is currently being incorporated into the Capital Improvement Plan for improvement in the next several years. Both these projects highlight the District's ongoing effort to continuously improve by evaluating its system, ensuring it meets its service goals, and taking action.

EARTHQUAKE PREPAREDNESS - LEARN FROM THE PAST

CCWD invests in continuously planning for emergency response. It has experienced several smaller emergencies over the past several years, but has not had to respond to a significant event. CCWD has, however, studied the response of others for major events such as the California Northridge and Loma Prieta earthquakes, and more recently the Hurricane Katrina disaster in Louisiana.

Each disaster has its own set of complexities. CCWD's emergency response plan ensures flexibility to most effectively prepare for various types of disasters and conditions. In studying past disasters, CCWD has identified the following key difficulties in response. The following section of this report provides some guidelines for how CCWD has prepared for these difficulties.

- <u>The ability to communicate is greatly reduced</u>. Almost all disasters place a significant strain on communication infrastructure. Not only is there the direct impact of the disaster on facilities such as utility poles, wires, underground infrastructure, towers and electronics, but there is the added "crisis" communication demand on the systems. To assist mitigate this impact, CCWD invested in significant person-to-person radios (walkie-talkie), satellite phones, as well as a County-wide radio system to improve emergency communications following a disaster. In addition, CCWD seismically anchored all its key communications equipment, and implements the latest earthquake design standards in its new construction.

- <u>Contacts with outside support agencies and resources are difficult to establish</u>. In the above noted disasters, the United States Federal Emergency Management Agency (FEMA) was the primary federal support agency. In addition, local state Office of Emergency Services (OEM) support was available at the state level. The support from these agencies tends to be slow. Any emergency response plan must focus on self-sufficiency as reliance on outside agencies and resources is unpredictable. CCWD conducts regular emergency response drills focused on self-sufficiency, and often includes its neighboring utilities and local county and city government in these drills. CCWD has key emergency contact information for all its neighboring agencies and municipalities.

- Transportation systems are greatly restricted. Roads and bridges, railways, traffic signals, and vehicle inventory itself is almost always greatly impacted by disasters. In the event of an earthquake, many bridges will be damaged beyond safe use, thereby blocking normal thoroughfare. An emergency response plan needs to address how movement of people and supplies will be greatly limited, and consider this impact by pre-staging supplies and people in strategic locations. CCWD secures emergency supplies, including food, water and shelter, for its staff. It has developed a radio call-in network to allow communication and response throughout the service area, even if transportation ways are blocked. In addition, CCWD is in the process of implementing a seismic valve improvement project that is installing isolation valves on all large transmission mains that cross key transportation corridors. The goal is to ensure that CCWD's facilities do not add to the impact of constrained transportation by flooding key transportation intersections and corridors.

- Power distribution is unreliable. Similar to the damage to communications, power distribution through poles and underground is vulnerable to many forms of natural and man-made disaster. Loss of power for a long period of time needs to be a part of response planning. In addition to the impact of power loss to water facilities, the impact to living conditions such as lighting, heat and ventilation, computers and communication equipment must be considered with any response plan. CCWD has installed emergency back-up power at all its key water treatment and pumping facilities to ensure a backbone emergency water supply system. In addition, CCWD has installed emergency power for its key administration facilities to ensure that response staff will have safe working conditions during an emergency.

- Monetary supply is greatly reduced; cash is limited. Banks and financial institutions are closed in the early stages of the emergency response. An effective emergency response plan needs to consider the ability to transact for services and supplies in the absence of electronic financial institutions and services. CCWD has emergency response contracts in place with all its key suppliers. The contracts commit to emergency supplies regardless of monetary system failure. Additionally, CCWD keeps a significant cash supply, secured, for emergency response.

- Commerce is greatly reduced. Not only is there the direct impact of materials, supplies and distribution (stores and transportation) being damaged and destroyed, but there is a shortage of individuals willing to conduct commerce. Individuals provide immediate support to their families. The result is that even if commerce has the infrastructure in place to function, staffing and other resources necessary to conduct business are greatly reduced. CCWD has developed staff deployment plans, has in-place standing orders for staff emergency response, and has also secured emergency response contracts with key contractors. CCWD in addition is prepared to provide limited staff family support as needed to ensure CCWD staff will be able to perform their duties, as well as meet their family needs.

- Safety and Security is a greater concern than normal. Unfortunately, though most people respond in a time of crisis by greater generosity, caring and giving, some individuals look at disasters as an opportunity to prey. The emergency response plan should keep safety and security in mind. CCWD has invested significantly in security and safety, especially following September 11, 2001. The CCWD facilities are all fenced and secured. In addition, CCWD places high priority on safety at all times, and also ensures that emergency response training includes a specific focus on maintaining safety standards under the pressure of emergency response.

Using these impacts as guidelines, the District has prepared a plan that addresses the impacts, yet allows for flexibility necessary to respond to variations.

THE RESPONSE PLAN

As with any plan, an emergency plan needs to be prepared with the end purpose in mind. The purpose of an effective emergency response plan is to provide the necessary framework to direct an effective response to emergency situations. CCWD's plan is not only the guideline for actual emergency response, but also the template for developing and facilitating training and practice.

Plan elements:

- Purpose of the plan clearly defined;
- Definition of key resources (staff, supplies, equipment, including checklists);
- Definition of key responsibilities of responders
- Definition of expectations for responder preparation
- Response staging areas clearly defined;
- Organization chart;
- Any necessary forms, or standard procedures (for example building inspection procedures and forms).

Knowing the variety of difficulties likely to be faced in an emergency (as identified above), the plan should include consideration of the following objectives.

- Be prepared to be self-sufficient
- Develop redundant, reliable communications
- Be prepared for a reduced work force - employees will take care of themselves and family first
- Pre-plan compliance with local, state and federal response requirements and guidelines
- Pre-plan business system continuity – electronic commerce will not function or be accepted

TRAIN AND PRACTICE

One of the significant benefits of any emergency response plan is its value as a foundation for training and practice. The plan not only becomes a solid starting point for training and practice, but then can also be the memory of the practice exercises through continuous modifications and updates. CCWD uses its emergency response plan for the foundation of all its exercises. Lessons learned from the practice sessions are recorded in the plan as modifications, and the plan becomes continuously updated.

Each agency and each event will clearly require its own specific response. However, CCWD has found that there are a few key aspects of training and practice that should be included in development of exercises.

- Develop practice scenarios that simulate likely actual conditions during the emergency response
- Practice actual deployment of resources
- Keep mapping and reference information organized, easily accessible and current
- Keep phone lists current
- Conduct actual table-top exercises on a regular basis
- Strengthen key team members through exercises specific to these team member responsibilities
- Lessons learned from each exercise should be defined, summarized and recorded
- Keep the emergency response plan current. The plan should be updated at least annually, and all key lessons learned from exercises should be incorporated into the plan.

While practice provides the best approach to simulating the actual emergency response, improving the skills of responders, and learning lessons for future improvement, training outside of practice is also important. In looking for training opportunities, training should include:

- Study recent disasters and responses
- Send key response team members to assist in response of actual emergencies, battle test the team
- Ensure lessons learned by these response team members are incorporated into the emergency response plan
- Develop contacts outside the area, and send team members to these contact training exercises, seminars and conferences
- Attend federal, state and regional training opportunities, and ensure the latest information is incorporated into the emergency response plan
- Ensure key managers are trained in standard response approaches
- Bring experienced responders and trainers to assess the emergency plan, develop exercises based on this plan, and oversee practice sessions.

Through continuous practice and training in the use, development and revision of the emergency response plan, CCWD ensures it will be as prepared as possible for the actual emergency. By looking outside the organization as well, including developing alliances, the agency can ensure it makes use of the most recent knowledge available for effective response.

STRATEGIC ALLIANCES FOR EMERGENCY RESPONSE

In the San Francisco Bay Area there are several water utilities providing water service to over 7 million people. While many neighboring utilities have coordinated for future emergency response for decades, and as noted above CCWD has informally coordinated with its local agencies, there has only recently been a concerted effort to coordinate emergency response on an official regional level.

In 2009, CCWD, East Bay Municipal Utility (EBMUD), Santa Clara Valley Water District, and the City of San Francisco, as part of a Water Research Foundation (formerly American Water Works Association Research Foundation) project, conducted a Regional Collaboration emergency response exercise. The objective of the exercise was to identify synergies and opportunities to better coordinate emergency response to ensure not only an effective response, but a more cost-effective response.

The Bay Area Regional Collaboration focused on identifying opportunities to better plan workforce, system response (for example through water system interties or shared facilities), contracts, and training. In January 2009 the agencies held a large scale emergency response exercise. The exercise included identifying and sharing respective response plans, and working to identify opportunities to learn; practicing scenario response between agencies as realistically as possible; and identifying potential mutual aid collaboration, and then establishing ongoing goals and follow-up actions to build an ongoing collaboration for the future. The ongoing actions included identifying logistics and distribution of resources that can be shared and coordinated, sharing of emergency response operations centers, and coordinating public information and warnings. The agencies continue to develop these improvements.

The key lesson of the collaboration is that large-scale disasters impact multiple agencies, and collaboration therefore allows for a large-scale, optimized response. While each agency can respond individually, a more effective, higher value, faster response can be provided if agencies plan a strategic emergency response that includes the diverse aptitudes and resources available. In addition, because most disasters have a varied impact (in other words, some areas are more impacted than others), regional collaboration can optimize the overall area's resources, with the result that the hardest hit areas receive a proportionately greater response.

The Bay Area agencies identified above continue their collaboration for emergency response. In addition, CCWD has outreached to several neighboring agencies and municipalities to assist them in emergency response. CCWD has partnered with many of its neighboring agencies to construct water distribution system interties (both untreated and treated water), and has mutual aid agreements with many of its neighbors as well. Emergency response planning needs to be ongoing, and CCWD continues this effort as well as part of its overall strategic approach to emergency preparedness.

CONCLUSION

The threat of disaster is present for almost every major urban area in the world. The result is that almost all utilities face a future need to respond to an unplanned emergency event. CCWD has taken a long-term, strategic approach at planning for disaster recovery. Its plan includes major capital investments to strengthen its core emergency response facilities, continuing investment in new and existing facilities using the latest design standards, preparation and training for emergency response, and strategic response alliances with local agencies.

CCWD will continuously review and update its emergency preparedness approach. It realizes its past good work, and good fortune to not have experienced a major emergency, provides little assurance for effective emergency response in the future without sustained attention to preparedness. By continuing its strategic view of emergency response, and continuing its overall investments and actions with this approach as its base, CCWD can be assured it will optimize its performance when the time comes to respond to any emergency.

REFERENCES

[1]. Contra Costa Water District, "Water Facility Supply and Reliability Improvement Strategy – Development of Seismic and Reliability Criteria Final Report – Volume 1," August 1994

[2]. Contra Costa Water District, "Seismic and Reliability Improvements Project, Volume 2", September 1996

[3]. Contra Costa Water District, "Seismic and Reliability Improvements Project, Volume 3", September 1996

[4]. Contra Costa Water District, "2002 Treated Water Master Plan Update, Final Report", December 2002

[5]. Contra Costa Water District, "Hurricane Katrina Disaster Response Assistance and Recovery," June 2006, AWWA 2006 Annual Conference and Exposition

[6]. Contra Costa Water District, "CIP's", February 1997 to 2009

[7]. Contra Costa Water District, "Seismic Evaluations for District Buildings", Simpson Gumpertz and Heger, July 2009

[8]. Contra Costa Water District, "Emergency Operation Plan", April 2008

[9]. Contra Costa Water District, "Engineering Response Plan", December 2008

[10]. WRF Tailored Collaboration, "Bay Area Water Utilities Operations Collaborative: 2009 WRF Tabletop Exercise After-Action Report/Improvement Plan", May 2009

Anti-seismic Measures of Water Supply Utility in Great Taipei Area

Pei-Chung Hsu[1], Yu-Chih Chou[2], and Yung-Ming Wang[3]

ABSTRACT

This paper presents the measures of emergency water supply against earthquakes in great Taipei area. Taipei city is the capital of Taiwan, one of the areas where earthquakes happen most frequently. After the devastating Chi Chi earthquake, many of water supply systems in central Taiwan were heavily damaged; the water department of Taipei city government attends austerely to the risk of water supply damages in great earthquakes. The water department has three anti-seismic measures nowadays: water supply system strengthening plans, emergency recovery measures, and emergency water supply measures. The emergency water supply countermeasures include emergency water tanks, emergency underground water supply plans, water supply trucks, and so on. We hope that with the introduction of this paper and through its discussion will lead to continuous collection of better advices to improve the area's anti-seismic measures of water supply utility.

Keywords : Anti-seismic Measures; Disaster Countermeasures; Emergency Water Supply

INTRODUCTION

921 devastating earthquake caused mega outage of water supply for more than 2 months before restoration. 2002 pan-Taiwan drought caused water ration in many areas in Taiwan and a 2-week water supply suspension due to typhoon in 2004 and 2005 in Taoyuan have prominently demonstrated the major difficulty of the public in acquiring safe drinking water and again drew government and the public's attention to the importance of water resources.

It is expected that the difficulties faced by tap-water supply will be more and heavier increasingly. Without strengthening hazard response capability will not be able to deal with the impact and challenge in extreme situation in the future, and building stable and safe tap-water supply system is closely related to the life line of the country as well as its sustainable development. Among various disasters, earthquake resulted to heavy and wide destruction to the tap-water supply system and therefore is affecting the water consumption of the public tremendously. The paper will discuss the present Earthquake Emergency Measures of Taipei Water Department (TWD) and it is hoped to be helpful in the hazard prevention and minimization of tap-water supply system in Taiwan in the future.

[1]Pei-Chung Hsu, Secretary General, Chinese Taiwan Water Works Association, 7F., No. 106, Sec. 2, Chang-An E. Rd., Zhongshan District, Taipei City 104, Taiwan
[2]Yu-Chih Chou, Assistant, Chinese Taiwan Water Works Association, 7F., No. 106, Sec. 2, Chang-An E. Rd., Zhongshan District, Taipei City 104, Taiwan
[3]Yung-Ming Wang, Engineer, Taipei Water Department, Taipei City, No. 131, Chang-Hsing St., Taipei City 106, Taiwan

STUDY OF THE HAZARDS

Hazards to tap-water is generally referred to damages brought to Tap-Water System by natural disaster or war that featured with wide scope of damage, large number of people depending on the water supply and long lasting effects. Tap-water supply incidence is comparatively referred to smaller scope of damages-less people affected and lasted shorter period. The later may be divided into natural and non-natural incidences. Natural incidence is, for instance, thunder strike or similar and non-natural incidence includes construction damage, fire, error in connection, improper operation and other artificial factors that cause to suspension of water supply. To distinguish hazard and incidence could be based on the level of damage and the scope of impact. The frequently-seen hazards and incidence to tap-water supply include earthquake, drought , typhoon, war, mud flood, water contamination / pollution, terrorism acts, facility incidence and construction accident. (Fig. 1)

Following global drastic change of weather, the hazardous incidences in Taiwan augmented in the aspects of dimension, frequency, diversity and complexity. Other than natural and environmental factors, human factors are not absent, such as urbanization, social development and deterioration of environment. According to the National Disaster Hotspots; A Global Risk Analysis, 2005 of World Bank, in Taiwan, 73% of its territory and population is exposed to 3 or more natural disasters and 99% of land and residents is facing 2 or more natural disasters.

COUNTERMEASURES AGAINST EARTHQUAKE

With the present engineering techniques and financial resources, it is impossible keep Taipei metropolitan area from partial suspension of water supply, shut-down or disability of water supply system when tremendous disaster in similar scale of 921 earthquake occurs. Therefore, the most important mission of TWD at present stage is to seek actively for most suitable technology with limited resources to be ready for hazard prevention or minimization of earthquake influences.

The countermeasure of TWD against tap-water supply hazards caused by earthquakes contains 3 main parts: Strengthening preparation of facility system, Emergency water supply measures and Aftermath restoration. The later two parts, Emergency water supply measures and Aftermath restoration may be categorized into Aftermath response actions. The interactions among the three are depicted in Figure 2.

The main part of countermeasure in dealing with earthquake hazard is to strengthen the facility system. The higher the facility system safety is, the lower the probability of being damaged will be. The facility system strengthening goals of TWD include: Diversified Water Sources, Sufficient Redundant Purification Plant Capacity, Sufficient Distribution Pond Capacity and Properly Positioned, Multiple Supply Pipeline or Circuit, Mutual Support of Trunk Line, Mutual Dispatch and Support between Neighbouring Supply Area, Individual Facility Equipped with Sufficient Redundant capacity, Mutual Support Capacity among Facilities.

Among the 3 parts, Emergency water supply is targeted at ensuring water source meeting emergency needs after the hazard. The strategies include: Building reserved emergency water source (incl. possible stored water within existing tap-water system, surface water, ground water, even the recycled water), stocking temporary water supply machine and equipment, emergency water supply at large medical institute and hazard shelters. The final critical target for the overall preparation is ensuring daily per 3~20 liters basic life supporting water supply (20 liters including basic hygienic water supply) per capita after the occurrence of most serious earthquakes estimated. (Fig. 3)

The establishment of emergency water supply facilities of TWD was divided into 3 stages. Stage 1 is adding life supporting emergency water supply facility with the existing distribution ponds and transmission trunk lines. It is expected to complete emergency life supporting water supply stations at 30 possible locations for storing water within the existing tap-water supply system. Stage 2 is to build quake resistant storage tanks especially for fighting hazard in 12 main urban disaster relief parks in Taipei City and Stage 3 ,a medium and long term plan, is to build spare water sources outside tap-water system by utilizing ground water or recycled water.

Most important thing in aftermath restoration is to complete the restoration promptly. In order to build up and ensure the capability of restoration, it shall strategically include the establishment of Hazard Commanding System, establishment of supporting system, unifying material specification, reserve of hazard machine and material, assurance of aftermath communication ability and prior reasonable restoration plan.

RESULTS AND DISCUSSION

1. Difficulty: Lack of compulsory statutory requirements for building quake resistant hazard preventing facilities; people tends to forgetting the scenario of hazard caused devastating earthquake, as time goes by, which makes it the first one to be deleted from budget at the difficulty of government finance.

   Program proposed: Under overall hazard preventing structure, to make law required for compulsory building of related facilities.
2. Difficulty: Despite it is well understood the necessity of building up emergency water supply facilities such as hazard preventing safety shelters, without encouragement acts– as domestic water is inexpensive, as well as it's not compulsory, it is difficult to request administration to build hazard shelters (park, school, hospital, etc) and to build out-of-tap-water-system reserve water source (quake resistant storage tank, emergency water well, recycle system, etc)..

   Program proposed: Providing economic incentive for building hazard preventing water sources by adjustment of water prices to a reasonable level.
3. Difficulty: Difficulty in obtaining new water sources – If we assume that water source dispatching program of Great Taipei Area is supporting Ban Shin Plant Supply Area with water source of Shindian River in one-way pattern, the probability of water shortage in the supply area of TWD will be shortened from once in 15 years to once in two years.

   Program proposed: In the midterm and long-term national land planning for a sustainable development of national land, the coordination of water resources and prevention and minimization of natural disasters shall be considered as well. In short-term water source dispatch, other than supporting the apparent shortage areas at speed; and an inclusive water supply system shall be established as well. In Great Taipei Area, it shall integrate 2 or 3 river systems into a multi-water source and inter-supporting single supply area is technically feasible and a permanent resolution at present stage.
4. Difficulty: Destruction of facility structure and pipeline are the main causes of tap-water supply interruption. Although the transmission and distribution pipeline of TWD is currently using flexible fitting DIP with high strength, high flexibility, impact and corrosion resisting; and the flexible fitting is featured with expandable and bendable for absorbing the force on pipe walls due to soil erosion, but once earthquake comes, the pipe and fitting located at week soil will not be exempted from the damage caused by uneven subsidence of soil.

Program proposed: Taking the supply of quake resistant piping materials, such as DIP pipe and fitting. It is suggested to replace the pipes and fittings with better quake-resistant piping materials (such as quake resisting DIP or HDPE) at locations with week soil (soil coefficient N lower than 10), sudden geological changes, fault, sliding slope, in high soil liquefying potential area and those place where the soil and the pipe wall may generate relative motion and create additional stress or concentration of strain, and adopt expandable flexible pipe. The advantage of quake resisting fitting is that the fall-off resistance of single DIP quake resisting fitting is 150 times of DIP flexible K-type fitting, and it has the chain locking effect and is capable of boosting the flexibility and quake resistance of pipeline.

CONCLUSIONS

Taipei is located in Circum-Pacific seismic zone and thus having occurrence of earthquake is natural and normal, only that it's unpredictable with current human-developed technology when it would occur and how its extent would be. Tap-Water is the necessary substance for life of people and is an importance resource indispensable to modern city and residents. When tap-water system is damaged by earthquake, the impacted scope is extensive, other than loss in direct damages, the normal life of resident will be affected and the urban life function could be paralyzed if the restoration is not completed in short time, and it may lead to secondary hazard.

We all know, the devastating earthquake is not avoidable in Taiwan Area, and in order to strengthen response capacity of tap-water system toward earthquake, in addition to establish complete hazard protection and minimization system in conjunction with hazard prevention and rescue units, water utilities shall build complete and proper water supply pipeline network, promote the adjustment and response ability in water supply , so as to control the scope of damage to keep it from expanding and / or deepening after occurrence of earthquake, or secondary hazard.

REFERENCES

[1] Measures for Increasing Spare Capacity for Future Water Supply in Great Taipei Area, Taipei Water Dept., Nov. 2005

[2] Sinotech Engineering Consultants Ltd., Subsequent Plan for the Water Supply Project of 5th Construction of Taipei Area, Taipei Water Dept., June 2006

[3] Y.M. Wang, Study on the Emergency Life Supporting Water Supply after Serious Earthquake in Urban Area, Chinese Taipei Water Works Association, Nov. 2005

[4] Shih Bang-Chu, Emergency Countermeasures for Life Supporting Water after Earthquake, Chinese Taipei Water Works Association, Nov. 2005

[5] G. T. International, Taipei Tap-Water Supply Pipeline Network Improvement Project, Taipei Water Dept., 2004

[6] Well Meet Technology Co., Ltd., Planning Report on the Study of Taipei Life Supporting Pipeline System Planning and Evaluation, Taipei Water Dept., Nov. 2003

[7] Sinotech Engineering Consultant Ltd., Report of Assessment Project for Earthquake ResistingDistribution Pond, Taipei Water Dept., 1998

[8] Wu Yang-Long, Study on the Prevention and Rescue Measure in Tap-Water Hazardous Incidence, Taipei Water Department, 2003

[9] Y.M. Wang, Plan for the Project of Using Existing Transport Trunk Line as Emergency Life-Supporting Water Supply Station, Taipei Water Dept., Apr. 2005

[10] C.T. Cheng, Study on Rational Dispatch of Water Supply System and Crisis Management in Taipei Area, Taipei Water Dept., 2005

[11] Wu Tian-Huan, Study on Emergency Response to Hazard of Water Utilities, Chinese Taipei Water Works Association, 2000

[12] Chen Yao-Nan et al, Analysis of Damages to the Water Supply System Resulted from Chi Chi Earthquake, Chinese Taipei Water Works Association, June 2003

[13] Wu Wen-Chuan, Evaluation of Earth Quake Damage on Taipei Tap-Water Pipeline System and the Emergency Measures, Dept. of Construction Engineering, National Taiwan University of Science & Technology, 2003

[14] Shih Bang-Chu, Study on the Preparation for Public Construction Hazard and Rescue Mobility System, Public Construction Commission, 2000

[15] Fundamental Plan for Hazard Prevention & Protection, National Disaster Prevention & Protection Commission, Aug. 2003

[16] Taipei City Area Disaster Prevention and Protection Plan, Taipei City Government, 2002

[17] Taipei City Disaster Prevention Operation Manual, Taipei City Fire Dept. 2004

[18] Shao Jiang-Bi, Study on the Urban Disaster Prevention Planning and Improvement Management Plan of Local Level Governments. – Urban Disaster Shelter System Planning Plan, Architecture and Building Research Institute, Ministry of the Interior, 2002

[19] Water Pipeline Techniques Research Institute, Pipeline Quake Resisting Plan Guidelines, Pipeline Construction Section, Pipeline Environment Division, Life Health Department, Ministry of Health, Labour & Welfare, Japan, 1997

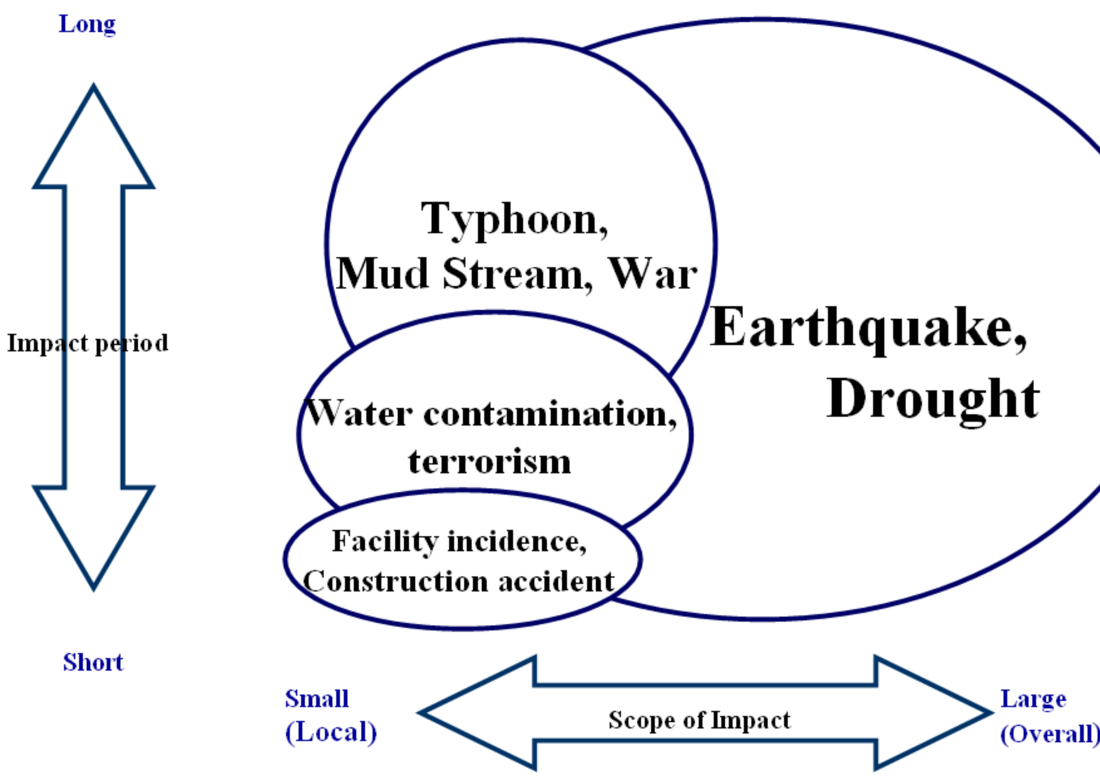


Fig. 1 – Diagram of Impact on Water Supply Resulted From Hazards

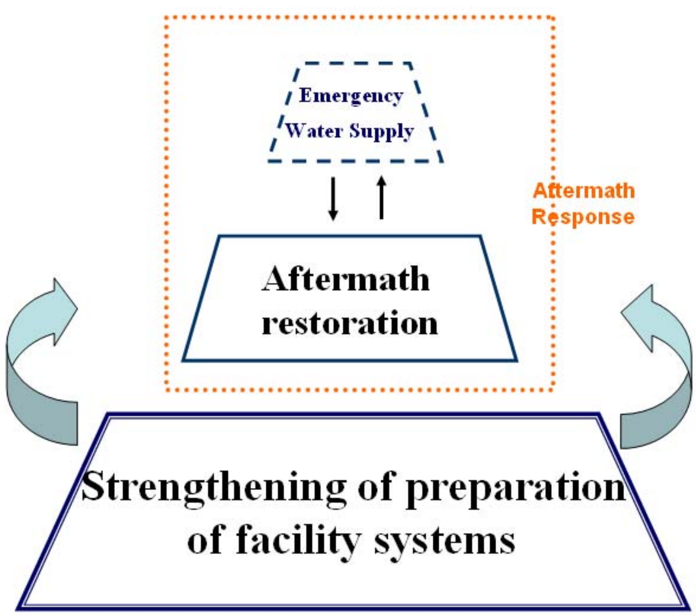


Fig.2 - TWD Water Supply Hazard Preventing Structure

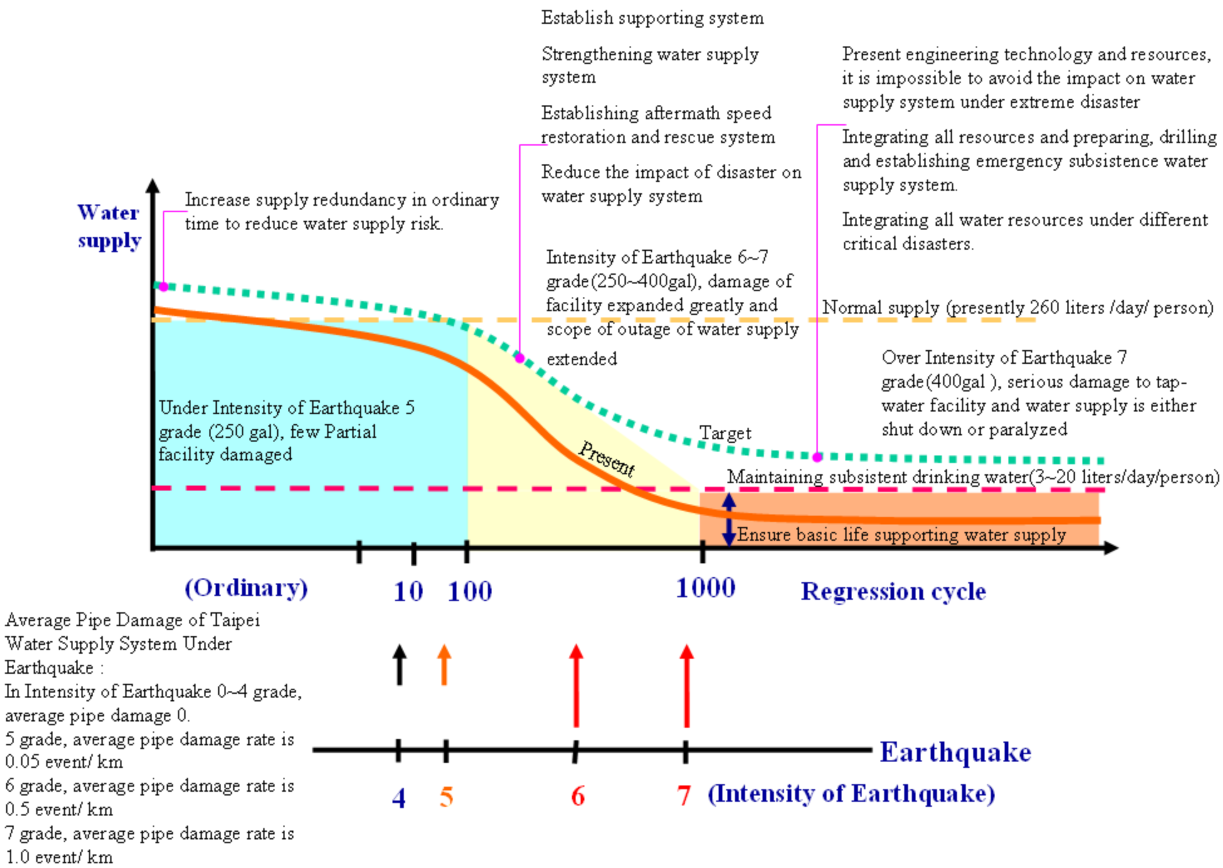


Fig. 3 - Diagram of Taipei Water Supply under the Regression Cycle of Earthquake

The Present Status, Problems and Promoting Procedures for the Measures Against Earthquake of Water Supply Facilities

Munetaka Abe and Yasushi Taguchi

ABSTRACT

There are four large scale earthquakes which are registered an intensity of 6 to 7 on Japanese scale after the Grate Hansin Awaji Earthquake. The major damages to water supply facilities were related to pipes caused by ground liquefaction near river bank and seaside area and by the ground transformation in mountain area.

On the aspect of water supply system, there are many problems such as the isolation of villages in mountain area, the long time lag to get information of their damaged facilities, the shortage of water supply engineers and the poor cooperation between utilities and contracted operators.

The Japan Water Works Association (JWWA) investigated the problems of hardware as the reinforcement of facilities and software as the preparation of emergency restoration procedures. Those should work closely together as an anti-earthquake measures and have close relevance and are complement each other.

This paper reports the present status, problems on reinforcement of facilities and inhibiting factors according to the survey and the way for promoting the measures against earthquake of water supply facilities.

1. THE ORIGINAL CONDITIONS AND ISSUES OF COUNTERMEASURES AGAINST EARTHQUAKES IN JAPAN

The Water works in Japan, many water facilities were constructed intensively for covering the water demand increased sharply during from 1955 to 1965. The period was called rapid economic growth. In the future these facilities are estimated to be older at one time. So it is worried to save the fund and declining the function of water facilities.

The water works in Japan are almost small or middle in scale, that has about 50,000 water supply population, in this case, it is taken as a matter of course that the water works has less than 5 engineers. It is not rare that there is no one qualified with engineer. It shows that the water works in Japan has a lot of issues owing to administrate the municipality in principle.

Concerning in conducting the water works facilities, regardless of the size of the water works, it is important that the function of the facilities are exercised in system. So the functions in state of earthquake occurred are influenced to the individual condition with locality and environment.

Munetaka Abe, Assistant Chief, Engineering Division, Japan Water Works Association (JWWA), 4-8-9, Kudan-minami, Chiyoda-ku, Tokyo, 102-0074 JAPAN

Yasushi Taguchi, Director of Engineering Department, Japan Water Works Association (JWWA), 4-8-9, Kudan-minami, Chiyoda-ku, Tokyo, 102-0074 JAPAN

Because of the old and rotten facilities advanced more quickly, it is most important issues to make the best plan for reconstructing the facilities systematically.
Regarding of the earthquake-resistant constructive condition of main facilities, the filtration plants are on the level of 13 %, the distribution pipes 23 %, main pipes 12 %, that shows extremely low level. On the other hand, both waterworks and water wholesale utilities have made the plans concerning anti-seismic measures, with the view to the water supply population, that reached to 70 %, but that is less than 10 % seeing with the view to the numbers of waterworks. It proves that most of the small or middle waterworks having less than 50,000 of the water supply population did not make the plan for the anti-seismic measures. **Figure 1** shows this.
When we make the plans for more earthquake measures, it is very important to get comprehensive and cooperation with customers, planning upgrades to existing water works (in order to make them more earthquake proof) must utilize competent anti-seismic diagnosis quickly and use the PI (Performance Indicator) to assess the original condition of earthquake proof. On the base of PI, we must make the plans and explain the contents simply to the customers and city assembly.

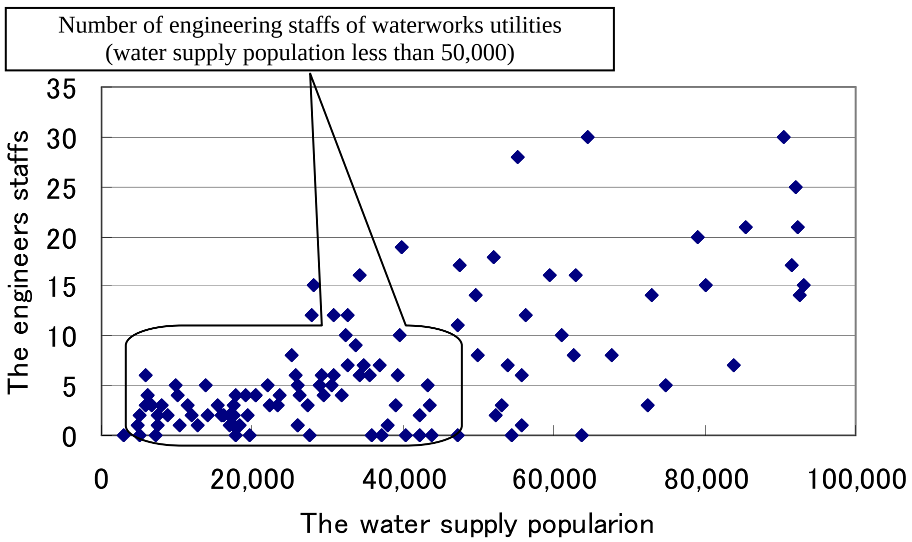


Figure 1 Water supply population and number of engineering staffs of small/middle scale waterworks utilities

2. PROVISIONS FOR ANTI-SEISMIC MEASURES BY JWWA

JWWA put together “The guideline of emergency relief measures after an earthquake” and “The issues and measures of restrain of water supply facilities after an earthquake” in accordance with that instruction, first document is the guideline regarding of restoration of emergency water supplies and how to contact, how to support with mutual action of water works after an earthquake. Second document shows the issues and the direction of anti-seismic proof analyzing the existed condition.

“The issues and measures of restrain of water supply facilities after an earthquake”

In order to consider the issue and direction of water facilities, JWWA carried out the researching the questionnaire targeting for 251 to be chosen at random from 14,000 waterworks utilities attached to chief JWWA members. As the result, most of waterworks in Japan recognize the necessity for anti-seismic measures, but then over half of waterworks utilities were delayed for planning the anti-seismic measures, particularly more than 60 % of small waterworks utilities did not made them. Main factors of these are difficulty of make the fund and insufficiency of engineer holding the skill of water technology. In order to solve these problems, it is necessary to gather the scale of other small waterworks utilities and then to reinforce the fund basis, in addition to good cooperation between the public agency and private sectors efficiently.

Specifically as concerning to advance the anti-seismic measures, I have analyzed from the three angles; “saving the fund for anti-seismic measures,” “promotion to comprehend the need of anti-seismic measures by the customers waterworks staff,” and “promoting the anti anti-seismic measures.”

2.1 The countermeasures concerning of securing the fund for anti-seismic measures

2.1.1 The revision of water tariff

Recently, Japanese economy tend to be deflation extremely, therefore another public utility charges have been cut the price being affected for global oil price movement, so it is very difficult to raise the water charges in order to get the fund for anti-earthquake proof in Japan. But it is principle for water utilities to propose the revision of water tariff because of the saving fund for anti-seismic proof. Utilities have to explain to the water customers in reference with the reason to raise the water tariff.

Regarding of questionnaire that was carried out by JWWA, 110 waterworks utilities proceed the change of water price from 2007 to 2008 and 41 waterworks utilities (37％) carried out the water price down. Besides most of waterworks utilities did not work on the anti-seismic proof for their facilities, in the future those facilities will be need to do the work for anti-seismic proof. Therefore budged deficit forced them soon to do the revision of water tariff for anti-seismic proof work. Anti-seismic proof does not give the water customers a big benefit rapidly, and does not link to the amount of charges. So it is very important to order the theory concerning the anti-seismic proof doing the public relations actively.

2.1.2 The application of the state subsidy

Regarding with the researching the questionnaire survey (182 water utilities) carried out by JWWA, the number of water utilities that used the state subsidy were only 5 utilities (2.7%). On the other hand, the other utilities almost have had strong interest about it. This mismatch shows that the ratio of the state subsidy is not enough for the expectations of water utilities. In spite of the financial difficulties it needs the emergency reform of the ratio of the state subsidy.

2.1.3 Another valid way of saving finance

Another way of saving finance are the application of the company bond, the money transferred of public accounting, the private bond (private finance initiative) expect by the revision of water tariff and the application of the state subsidy.

2.2 Promoting the comprehension for the anti-seismic measures to the water works staff

Water customers can not realize the value of water because they can use enough water every day. Therefore they think the issue for the other people matter, but they can not understand unreasonably the emergency issue by reason of that probability.
First it is the most important point that how the water customers realize the significance of the anti-seismic measures of the waterworks facilities. So it is important for the water utilities to use the visual images, newspapers, experiences. And more important thing is that the water utilities' engineers have to master the necessity knowledge and to become the key person of the promoting the anti-seismic measures of the water supply facilities.
The table below shows the promoting measures of the anti-seismic measures of the water facilities for both the staff of the water utilities and water customers.

Table 1 The promoting measures of realizing the anti-seismic measures

The Object	The promoting measures of realizing the anti-seismic measures
Staffs of the water utilities	Staff (1) offering of a plain documents to be well known the anti-seismic proof
	Staff (2) taking place for a workshop and meeting regarding of the anti-seismic proof by the all prefecture
	Staff (3) meeting each other by the staff having a earthquake to be supposed in common
Water customers	Water customers (1) a substantial HP and publicity
	Water customers (2) practicing of a trainings for disaster and delivery lesson
	Water customers (3) practicing of questionnaires monitor system in each water utilities

2.3 Measures of practicing of anti-seismic measures

Estimating from the result of questionnaire and hearing researches, the main suffocating factors were the lack of experience for anti-seismic measures and the lack of engineering staffs having a special technique. Also we assumed that small utilities to be influenced by these factors are attended to be late concerning the anti-seismic measures.
On the base of these facts, we have practiced an analysis of measures for promoting of anti-seismic measures.

2.3.1 Basic view of measures of preparing for anti-seismic measures

Measures of making the water facilities to be strong for the anti-seismic proof, we can separate the measures of anti-seismic proof into two ways. One of them is the measures of anti-seismic proof of water facilities and second is first-aid measures, First issues is the maintenance of facilities not to be broken away at earth quake happened, the purposes are control of damages and minimizing the influence of earth quake disaster.
On the other hand, first aid is the supplying minimum water to live at least in spite of facilities being damaged, the main purposes are quick recovery and securing the substantial quantity for emergency water supply.
Figure 2 shows the order of measures to be taken as anti-seismic measures.

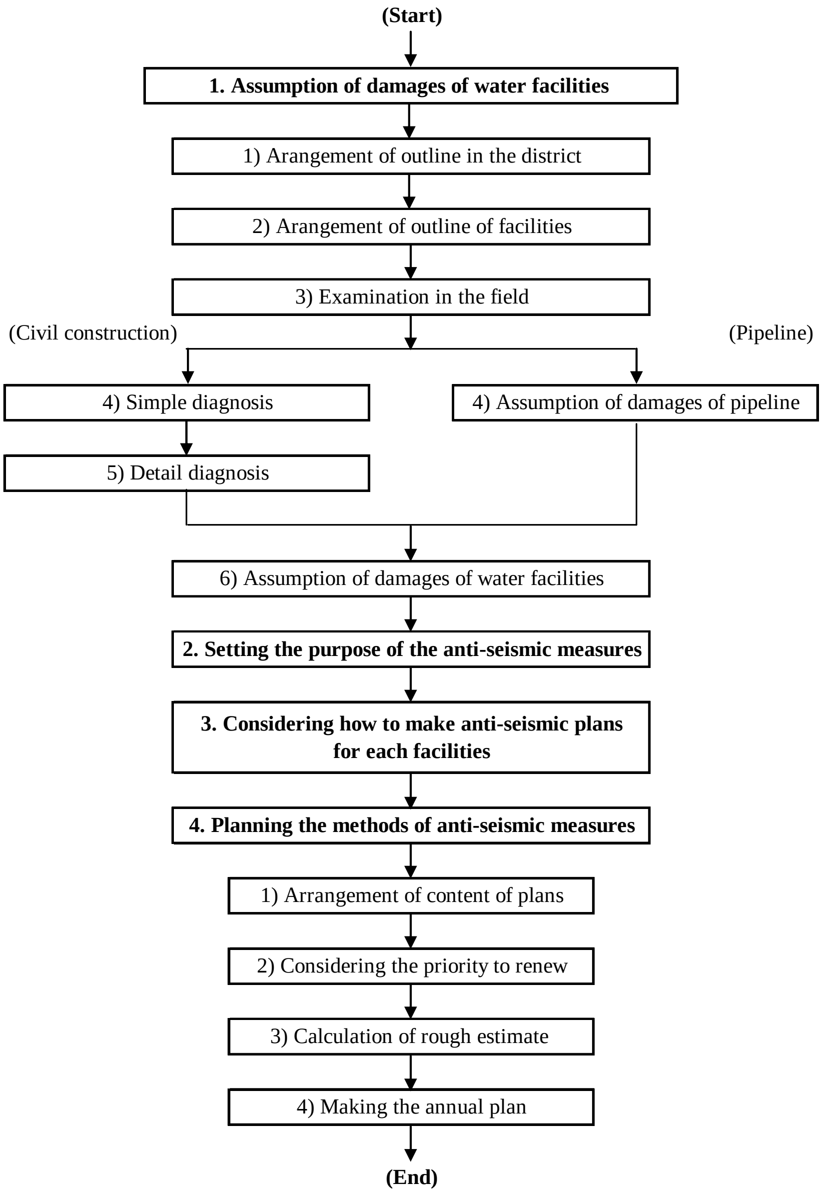


Figure 2 The practicing flow of anti-seismic measures planning

The item paid attention to the planning measures of anti-seismic proof consistency with another plans. All the measures must be planned in adequate coordination with the regional emergency preparedness plan and the plan of making town considering the typical regional condition.
Setting the target and practicing gradually considering the importance and emergency to accomplish the target, according to the manpower and fund it is important for the water utilities to make a practical plan.

- Control the process
  Water utilities have to control the process on the base of ratio of anti-seismic measures of the pipeline and facilities and show the arrival numbers.
- The freedom of information
  Water utilities have to show the arrival number, the process and the effect regularly.

2.3.2 Entrusting the job of anti-seismic measures

According to **Figure 2**, small water utilities have a lot of difficulties to practice the diagnosis, assumption of damages of water facilities and planning anti-seismic measures because of the lack of manpower. So it is necessary to entrust that jobs to the outside company.
The difficult facts and promoting measures are shown below;
1) Difficult factors to entrust the jobs to the outside company

a. Securing the fund to entrust
Most water utilities in Japan have difficulties to persuade the assembly to get the fund for the anti-seismic measures because of making little progress of the income rate of water

b. Lack of manpower
Because of the lack of the professional staffs, almost small water utilities have difficulties to make a specification, the job to entrust, inspection the report, having a skill to entrust, both office work and so on.

c. Promoting the job to entrust

- Maintenance of the document regarding of the inspection and so on
  It is assumed that main entrusting jobs are planning the measures of anti-seismic proof, practicing the diagnosis, the design of reinforcement of the facilities. But they are not served yet. So the examinations are needed quickly.
- Maintenance of the existent document
  The case of entrusting, it is necessary to grasp the contents and the amount of work, job abstract, easy document to study the facilities easily, to study the drawing and account
- PR from the receiver to entrust
  For the promoting the job to entrust, it is necessary to solve the factor with the agency to entrust as well as the agency (for example, consulting company) to be entrusted. It is very important to use the diagnosis chart of water facilities to review the present condition of the water facilities, and to present the new issues.

2.3.3 The techniques and methods of effective anti-seismic construction

The water utilities are asked from water customers to secure the preventing the water

leaks and safety of water quality in spite of new or renewal facilities to be constructed.
In the case of constructing water facilities for the anti-seismic reinforcement, it is necessary to consider the influence on water treatment, amount of water distribution, loss of water pressure, and water quality as well as studying of back up skill secured. On the other hand, in the case of being difficult to reinforce the facilities, it is needed to review the all water facilities pointing of view the water system.
Abstract of anti-seismic measures are shown below;

1) Earthquake-resistant pipelines under the ground

Earthquake-resistant pipelines under the ground are carried out to replace the old pipe with the pipes which have anti-seismic functions. But regarding the large scale earthquakes, many water leaks are expected in the several places, so it is necessary to maintain the earthquake-resistant pipelines as well as to make a proper water pipeline network and to maintain the double pipelines for backup of the main pipelines.

Beside if we compare total constructing cost of pipeline of diameter 150mm and 250mm having earthquake-proof joints such as NS with same diameter pipeline having joint like K type, NS type needs only 1% higher cost than K type joint. So the earthquake-resistant pipelines have advantages than not having earthquake-proof joints. We should appeal these effects like B/C (Benefit by Cost) to the water customers.

2) Anti-seismic measures for water facilities

There are many particular water facilities like the construction composed by line, for example, water way, cooperation ditch, shield tunnel and so on. These facilities have difficulty to find the damage points or to reinforce them. So it is important to consider the proper safety rate at the new-built. The constructions on the ground were not almost designed by matching to seismic Motion Level 2. After working the easy diagnosis to the facilities, it is significant to reinforce the water facilities evaluating the seismic performance on the base matching to seismic Motion Level 1 and 2.

3) Anti-seismic proof of the aqueduct bridge

Concerning the reinforcement of the aqueduct bridge, the measures of anti-seismic proof are ordinary practicing to the superstructure because of the fact caused by the example of the recent earth- quake damages, the other hand, in case of underground construction it needs the proper consideration for earthquake damages by the cooperate motion or the movement motion with both bridge table or concrete legs of adequate bridge and ground. Particularly there are many rivers in Japan. So many adequate bridges were constructed. Therefore effective way of reinforcement to the adequate bridges are necessary with reference to TC (Total cost).

4) Anti-seismic measures of machinery and electric equipment

Regarding anti-seismic proof of the machinery and electric equipment, each one of them have to have resistance for the earthquake, strong fix or allowing the tremor of earthquake prevent the fail and fall. And enough length of wiring secures against the damages for earthquake of that equipment. And proper elastic pipe has to be used at the perforation of wall.

5) Improvement measures of preventive factors of using new technique or method of construction

On the way to improve the anti-seismic proof, it is desirous to develop the new technique or method because of the anti-seismic, construction, economy and so on.

Improvement measures of preventive factors of using new techniques or methods of construction are shown in **Figure 3**.

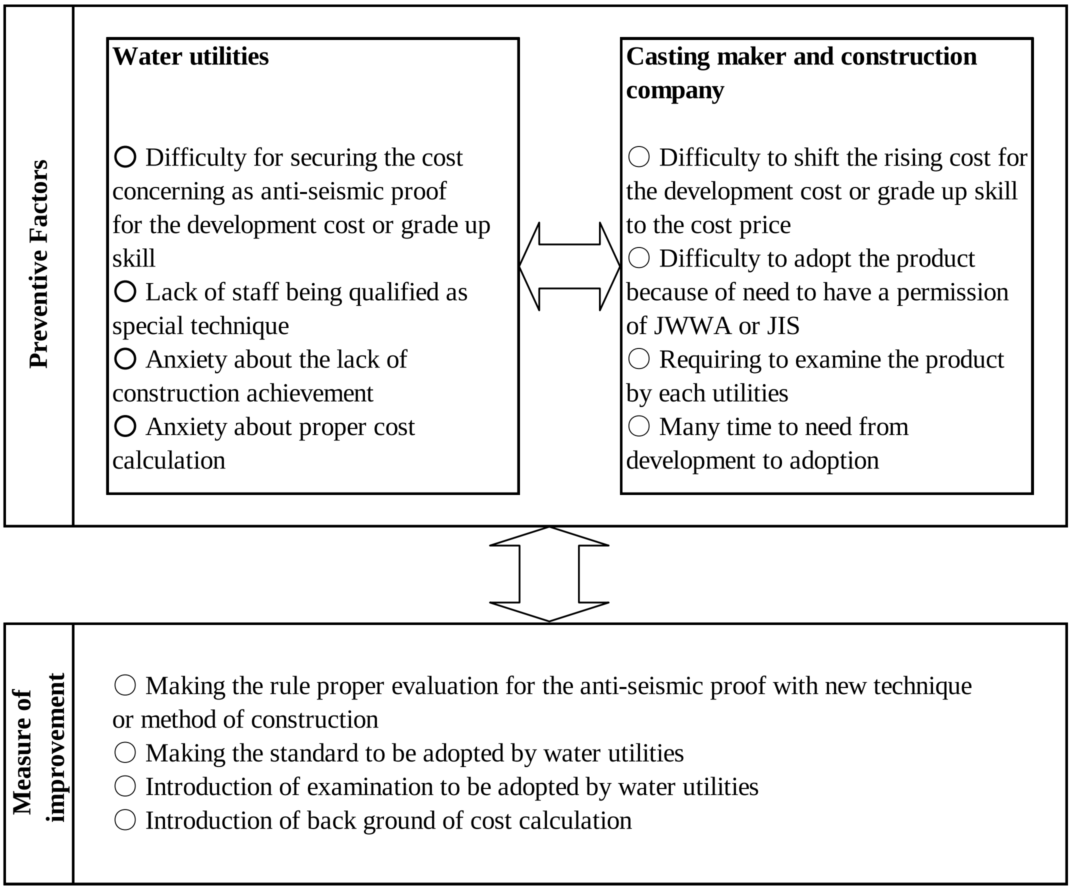


Figure 3 Preventive factors and improvement measures of new techniques and methods

2.3.4 Management of techniques for the improvement of anti-seismic measures (efficiency of construction clerical work)

The things regarding design and calculation (making a figure of design, calculation of structure, calculation of quantity, design of calculation) are called "management of technique." It is essential to do it exactly for advancing anti-seismic construction work smoothly and effectively.

1) Efficiency using the various systems

It is necessary to adopt the job to each specification and to make the standardization. There are problems such as ramification of constructions, lack of expert staffs and difference of designers skill etc., so various design/calculation systems are carried out to solve these problems.

2) Simplification using the mapping systems

There are a lot of utilities using the mapping systems to manage huge information on pipelines. Preserving and managing information of water pipes contributes the simplification of design work. And information on buried location of other pipes contributes the simplification too. In the future it will be necessary to exchange one water utilities' data for other's each other.

3) Economical practicing of construction of pipe

Due to the financial difficulties we have to practice the new technical improvement to reduce the cost of construction in addition.

The practicing example of economical constructions are shown below;

a. Review of laying depth of water pipes (laying pipes in shallow depth)
Change from DP1.2m to DP0.6m

b. Review of cross section of digging flame for water pipe
In the case of laying pipes in depth 1.5m, change from 63 degree against the cross section digging to straight digging

c. Reducing the influence width of fundamental ground
Applicable to thin pavement

d. Cooperation with other business company

e. Effective using of soil due to construction

f. Prier pavement construction
Practicing the laying pipes to cooperate with other road construction because of reducing the cost of pavement recovery

g. Construction of Pipe in Pipe (PIP)
The construction to insert the new pipe into the old pipe without digging, the cost of this construction is expected equal efficiency comparing with laying pipes with digging.

3. CONCLUSION

In order to keep either problems or issues of Japanese waterworks utilities in order, we discussed the preventing factors and the measures hereafter.
We can't conclude it by the ace spades. Making up for the lack of fund or staff, we have to combine with the options to solve the problems and issues efficiently.
Therefore it is necessary to extract the preventing factor by the flow chart of this report, to clear the background or the cause. Then we must practice the measures being possibility according to the flow chart to solve.
The business for the anti-seismic measures needs a lot of fund and long range term, so it is very important to comprehend the real meaning to the water customers and have to make a long-term plan for anti-seismic measures, according to a real condition for the lack of fund and staffs.
On the other hand, making a long range plan, we must start the emergency business (for example, the measures to exchange the asbestos pipes) prior to the other businesses.
If the emergency business is not practical, we will prospect for the serious damages of water facilities by the earthquakes, in fact there are many cases like that damages in Japan in the past.
I will have a great pleasure of this report to contribute the advance of anti-seismic measures for the small or middle waterworks utilities.

A Strong Community Outreach Plan Will Facilitate the Implementation of Complex Seismic Upgrade Programs

David L. Pratt[1] and David D. Lee[2]

ABSTRACT

The East Bay Municipal Utility District (EBMUD) was the first major water utility to implement a seismic improvement program. It used state of the art expertise in the areas of seismology and structural analysis to evaluate the vulnerabilities of its water system. EBMUD set goals for customer service and mitigating economic impacts, and then it developed a capital improvement program to attain those goals.

EBMUD developed a strong community outreach plan to educate and solicit input from the community, and then to gain community support for the proposed capital program. The outreach plan used a proactive, straightforward approach that utilized a variety of communication methods.

The public greatly supported EBMUD's proposed seismic improvement program. The community outreach plan worked so well that the public was not only strongly in support of the program, the public asked EBMUD to complete the program within a 10 year timeframe instead of the proposed 15 year timeframe.

This paper will discuss the approach utilized in the community outreach plan and the various communication tools that were used to gather input, educate the public, and gain community support.

INTRODUCTION

Engineers are trained to identify problems, analyze systems and generate alternatives as they resolve complex technical issues, such as how to mitigate for seismic events. The proposed solutions are often creative, ingenious and expensive.

Yet, after all the engineering is done, the project must often be approved by some type of governing body before it can be implemented. If the public is against the project, the governing body will be reluctant to authorize the project and implementation may never happen.

This paper will describe the communications program that EBMUD used to educate its ratepayers and achieve public consensus for its Seismic Improvement Program (SIP).

BACKGROUND

EBMUD is the nation's tenth largest water utility. As a regional public agency, EBMUD supplies water to more than 1.3 million customers in the eastern portion of the San Francisco Bay area. The 325 square miles of service area include 20 incorporated cities and 15 unincorporated communities.

[1] Manager of Design, EBMUD, P.O. Box 24055, Oakland, CA 94623, DPratt@EBMUD.com

[2] Senior Engineer, EBMUD, P.O. Box 24055, Oakland, CA 94623, DLee@EBMUD.com

The EBMUD's major source of water supply is the Mokelumne aqueduct system. The Mokelumne aqueduct system consists of three 82-mile long, large diameter aqueducts. It brings raw water from Pardee reservoir in the Sierra Nevada foothills to the local service area.

In addition, three terminal reservoirs, filled with service area watershed runoff and aqueduct diversions, supplement the aqueduct water supply to meet the service demands. Treated water from six water treatment plants is distributed to 167 storage reservoirs by gravity feed and/or pumping.

The service area of EBMUD is located in a region of high seismic activity. There are several major earthquake faults that intersect or are located near the EBMUD service area. In 1991, EBMUD began a two year evaluation of the seismic vulnerability of its water treatment and distribution system. The project team included EBMUD staff and consultants with expertise in the areas of seismology, water operations, structural analysis, and economics. The resultant study proposed a program to improve post earthquake service capabilities. Subsequently, EBMUD staff developed a Community Information Program (CIP) to inform the EBMUD customers about the proposed SIP and to gain community input and support. With favorable feedback from all segments of the community, the program was adopted in 1994 and completed in April 2007.

EVALUATION RESULTS

Even though existing EBMUD facilities were originally built in accordance with the best technology at the time of construction, much knowledge in the fields of seismology and earthquake engineering has been developed since their construction. Not surprisingly, the study indicated that the water facilities would be extensively damaged following a major earthquake, and water service would be seriously affected.

The key findings of the study were:

- EBMUD facilities would be particularly vulnerable to seismic impacts because of localized geological hazards, such as landslides, liquefiable soils, and numerous active faults, and because critical facilities are located very close to the active faults.
- Many essential facilities would be damaged in a major earthquake. The study indicated that, after a Hayward magnitude 7 earthquake, four out of the six water treatment plants would not have been functional, one-third of the distribution reservoirs would have suffered damage, and two-thirds of the pumping plants would have been out of service, either because of power outage or structural failure. Also, 5,500 pipe breaks would have occurred throughout the service area.
- A key transmission tunnel, Claremont Tunnel, would have been severely damaged in a major earthquake event, disrupting water service to 70% of the customers.
- Customer service and fire flows would have been impacted in all scenario earthquakes.
- Repair costs following a major earthquake were estimated to exceed $400 million (2009 dollars).

COMMUNITY INFORMATION PROGRAM

Explaining to customers how the EBMUD's water treatment and distribution system function and the inherent seismic vulnerabilities were very important. If the customers did not understand how the system worked and the risks to the system in a major earthquake, they would not be able to understand why EBMUD needed to strengthen its system. It would have been very difficult to get the public to accept a $350 million (2009 dollars) seismic program if they did not understand it.

The SIP was one of the largest programs ever undertaken by EBMUD. SIP staff initiated the CIP to educate the public about a vitally important program and promote community acceptance of EBMUD efforts to improve earthquake safety and response. Additionally, EBMUD wanted to obtain valuable feedback to assist its Board of Directors in making important financial decisions.

EBMUD staff utilized a straightforward process to communicate with the public. First, it focused on educating the ratepayers so they understood the issues. Then, to gain support, staff carefully explained the program's goals. Finally, staff gathered feedback from the ratepayers to create the final SIP.

Educate the Public

Educating the public about the water system vulnerability, program scope, costs and rate impacts was the crucial first step. Lack of public understanding of the program would have resulted in public resistance, and implementation and financing of the program would have been much more difficult. Staff explained several key concepts to the public:

- Current System Vulnerability – At the time of the original study, seismologists predicted a 28% chance that there would be a major earthquake on the Hayward fault (northern segment) by the year 2020. Most customers didn't realize how vulnerable the EBMUD's water treatment and distribution system was to damage in a major earthquake. They didn't realize that there would be thousands of pipe breaks and that two-thirds of the customers would be without adequate water service, some for up to six months, nor did they realize that water for fire fighting would be severely limited. The financial impact in the EBMUD service area was estimated to be as great as $3 billion in repair costs and business-related losses.
- Program Costs – The adopted program cost approximately $350 million in 2009 dollars.
- Rate Impacts – Staff explained the resultant monthly increase to their water bill and how it would affect the ratepayer personally

Gain Public Support

Once staff explained the vulnerabilities and the costs of the proposed seismic program, staff then explained the goals, such as faster return-to-service times and increased fire flow capabilities. Outreach to customers was essential to gain support for the program. The more information the customers had, the more they understood the goals and funding of the program, and the more they were willing to support the program. This minimized potential project delays due to public opposition.

A key reason the communications plan was successful was that the program goals were well-defined. The ratepayers understood what EBMUD was trying to achieve. Staff explained the program would not upgrade the whole system, but would focus on the parts of the system that are most critical to restoring service after a major earthquake. Staff also explained that the mitigations would reduce the extent of damage securing vital operating equipment, preventing major water losses and protecting critical pipelines, providing some redundancy and reducing post-earthquake recovery time.

Solicit Feedback

EBMUD obtained feedback to assist in making important financial decisions. EBMUD heard from more than 1,700 customers through letters, comments after presentations, and survey responses. Staff developed a simple questionnaire and ratepayers were encouraged to provide their opinions on keys program components such as:

- Program Scope;
- Duration;
- Willingness to Pay.

Each of these decisions had a significant impact on water rates, which was a sensitive issue with the ratepayers.

METHODOLOGY

The CIP was successful because it was proactive, straight forward, factual and open. The selected approaches generated good working relationships with the public and other agencies, and improved coordination efforts.

The CIP focused on two main components. First, it identified and targeted various audiences and the messages it wanted those audiences to receive. There were different goals associated with each audience. Second, it identified the tools that would be used to convey those messages. A broad range of tools was used to achieve the goals identified for each target audience.

Target Audiences

Staff identified four general audiences for outreach:

- General Public
- Internal Employees
- Technical Community
- Emergency Planning Coordinators

General Public

The goal in reaching out to the general public was, through education, develop and maintain support for the SIP. Staff determined that it could more efficiently implement the SIP if the community strongly accepted efforts to improve earthquake safety and reduce risk. Elected officials were included in this group.

Internal Employees

The goal for reaching out to EBMUD's internal employees was to maintain internal support. SIP staff determined that they would be able to implement the program more efficiently if all employees understood the need to improve earthquake safety and reduce risk. This helped keep resources focused on the high priority seismic program, plus EBMUD employees were better ambassadors for the seismic program with the general public.

Technical Community

Staff continually reached out to the technical community. EBMUD communicated with experts around the world and informed them about the SIP. This ensured that EBMUD had access to current seismic information and research that was state-of-the-art. EBMUD staff also obtained valuable feedback for its seismic mitigation designs by reaching out to the technical community.

Emergency Planning Coordinators

SIP staff described its program to emergency planning coordinators from other utilities so that emergency responses could be coordinated on a regional level. The information sharing was also used to prioritize projects.

Communication Tools

A variety of communication tools were used to communicate with the various audiences:

- Speaking Engagements and Slide Shows – EBMUD Board Members and staff conducted over 70 presentations on the SIP. More than 1,700 people heard directly about the program, with the opportunity to ask questions and receive additional information.
- Fact Sheets – Fact sheets were distributed to approximately 1,700 people at over 50 presentations. They were also distributed to the media at media briefings.
- Bill Inserts – Bill inserts were mailed to 356,000 customers with a brief set of questions that customers answered using a voice-mail number provided.
- News Letters – News letters on the SIP were mailed to 1,800 selected customers and 1,200 business customers.
- Letter to Opinion Leaders – Letters from the General Manager on the SIP were mailed to 300 community and business leaders.
- Press Conferences – Two press conferences were conducted. They were attended by reporters from 8 newspapers, 4 radio stations, 2 television stations and one engineering journal.

- Public Meetings and Workshop – Two public meetings and one workshop were conducted on the SIP.

The following tools were utilized to solicit and generate feedback.

- Questionnaires – questionnaires were handed out at all presentations. Approximately 200 questionnaires were returned.
- Response Cards – Customers used the EBMUD furnished cards to send in their comments.
- Workshop – a workshop on the SIP was conducted by staff on October 17, 1994 (Anniversary of Loma Prieta Earthquake, the 1989 earthquake that caused significant damage in the San Francisco Bay Area), and featured the status of utilities preparedness and actions for the "next big one." Over 70 people attended, representing public agencies, other utilities, emergency services, and businesses in the San Francisco Bay Area. The workshop generated a lot of interest, comments and questions from the attendees.
- Voice Mail Responses – The responses to a questionnaire via voice mail, in EBMUD's bill inserts, were far greater than expected. Over 1,200 callers answered questions on the EBMUD's proposed SIP. The bill inserts also generated approximately 300 letters in response to EBMUD's proposed program.
- Public Meetings – Two public meetings were held to provide additional opportunity for community information and input to the issues being considered in the proposed SIP.

The following communication tools were considered but not used in the Community Information Program because of the high cost, long production time, or limited impact.

- Video Presentations
- Phone Surveys
- Focus Groups
- Key Leader Interviews

Results of Community Information Program

The success of the outreach effort was evidenced by the overwhelming support of the program by EBMUD customers.

- 90% Approval – The proposed SIP received widespread support from all segments of the community reached by EBMUD's communications efforts. (Program support: 92% from voice mail; 85% from questionnaires; 91% from letters.)
- Numerous Compliments – EBMUD received numerous compliments from customers, public agencies, utilities, emergency services and businesses.
- Few Complaints on Process – Only a few complaints were received and they were mainly related to the proposed funding mechanisms for the program.
- Program Definition Encompassed Public Support – Input from the public was incorporated into the final scope of the program.
- Support, Not Resistance – The support for the program was overwhelming and there was basically no resistance from the public or local agencies.
- SIP Staff were strongly encouraged and motivated by the overwhelming support of the proposed program.

SUMMARY

In summary, a successful Community Information Program should include these essential approaches:

- Educating the public and soliciting feedback;
- Taking a proactive, open approach;
- Utilizing a variety of communication tools to educate the public and gather feedback;
- Continuing commitment to maintaining public awareness and support.

EBMUD’s continuing effort included newsletter updates, construction signage, line items on water bills, presentations to community groups and internal communications.

The extensive outreach effort by SIP staff resulted in overwhelming support for the program. The public’s understanding and support of the program facilitated program implementation and allowed EBMUD to more efficiently manage the SIP.

Activities Related to Earthquake-proofing of Drinking-water Infrastructure in the Japan Water Research Center

Mikita Amano, Yasuhiro Suzuki and Masahiro Fujiwara

ABSTRUCT

Japan is one of the most earthquake prone countries in the world. In recent years, M7-level earthquakes occurred in the prefectures of Niigata, Ishikawa and Iwate, after the Hyougoken-Nambu Earthquake in 1995. These earthquakes caused extensive damage to the drinking-water infrastructure. Therefore, earthquake-proofing, as well as renewal of aging drinking-water infrastructure, is one of the most crucial tasks for drinking-water utilities.

It is important not only to ensure a stable supply of water at normal times, but also to minimize the risk of accidents and malfunction, especially in case of disasters. Preventive maintenance of the facilities is an important aspect in this regard. In addition to conventional maintenance procedures, performance assessment to evaluate earthquake resistance should be implemented, in order to ensure that the facilities respond to the needs of society and its citizens.

The Japan Water Research Center (JWRC) has conducted various researches and developed methods and tools for drinking-water utilities to implement earthquake-resistance facilities. The first part of this paper shows the present situation of earthquake-proofing facilities in Japan using performance indicators (PIs). The second part presents a method for planning earthquake-proofing using the latest guidelines issued by JWRC. Finally, the paper will report on a simplified method of evaluating earthquake-resistance which is currently being created by JWRC.

THE CURRENT SITUATION OF EARTHQUAKE-RESISTANCE OF FACILITIES

In January 2005, The Japan Water Works Association (JWWA) defined 137 "PIs" as a JWWA standard to assess Japan's drinking-water utilities quantitatively. JWRC performed an analysis of these PIs in water utilities nationwide for the first time [2]. The analysis results on earthquake-resistance are shown below.

Mikita Amano, Japan Water Research Center
Toranomon Denki Bld. 2f, 8-1, Toranomon 2-Chome, Minato-ku, Tokyo, Japan, 105-0001
Yasuhiro Suzuki, Japan Water Research Center
Toranomon Denki Bld. 2f, 8-1, Toranomon 2-Chome, Minato-ku, Tokyo, Japan, 105-0001
Masahiro Fujiwara, Japan Water Research Center
Toranomon Denki Bld. 2f, 8-1, Toranomon 2-Chome, Minato-ku, Tokyo, Japan, 105-0001

Ratio of Earthquake-Resistant Treatment Facilities

Figure 1 shows the frequency histogram of the ratio of earthquake-resistant treatment facilities. This is defined as the capacity of earthquake-resistant treatment facilities divided by the capacity of all treatment facilities.

A great number of utilities have a ratio of earthquake-resistant treatment facilities of 0%, and the average across Japan in FY2005 is 12%. However, looking at the change in the earthquake-resistance ratio from FY2005 to FY2006 in Figure 2, an improvement can be seen slightly. In addition to the fact that treatment facilities are required to reach an extremely high level of earthquake-resistance (rank A against level 2 earthquakes) in Japan, earthquake-proofing will not be reflected in this ratio unless all equipment comprising the facilities is made earthquake-resistant; as a result, the earthquake-resistance ratio has not shown any great improvement.

$$\text{Ratio of earthquake-resistant treatment facilities (\%)} = (\text{capacity of earthquake-resistant treatment facilities} / \text{capacity of all treatment facilities}) \times 100 \quad (1)$$

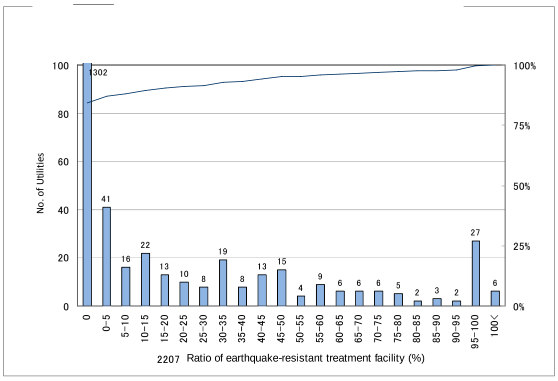


Figure 1: Ratio of earthquake-resistant (*1) treatment facilities (Number of utilities: 1,543)

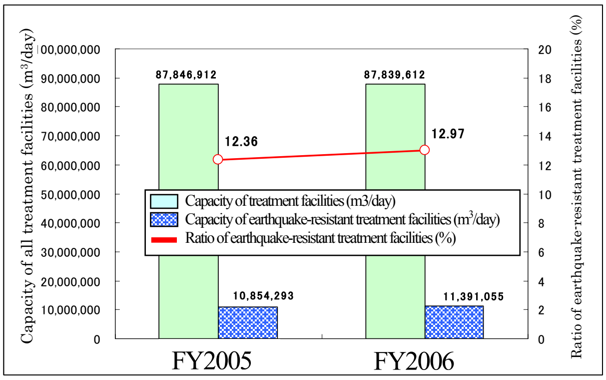


Figure 2: Comparison of the ratio of earthquake-resistant treatment facilities across two fiscal years

Table1: Level of Required Earthquake-Resistance

Degree of importance	Earthquake level: Level 1* i	Earthquake level: Level 2* ii
Rank A	No seismic damage	No injury to human life. Individual facilities are able to maintain their functions even when minor seismic damage has occurred there.
Rank B	Individual facilities are able to maintain their functions even when minor seismic damage has occurred there	Individual facilities are able to maintain their functions as total waterworks systems even after structural seismic damage has occurred there. Restoration is possible at an early stage.

* i: Level 1 : The maximum level of earthquake which may occur during the service period of the faciity

* ii: Level 2 : The maximum level of earthquake which may occur at the site of the facility in the future. Generally, level 2 is equal to or greater than level 1.

Ratio of Earthquake-Resistant Service Reservoirs

The frequency histogram of the ratio of earthquake-resistant service reservoirs is shown in Figure 3. This is defined as the capacity of earthquake-resistant service reservoirs divided by the capacity of all service reservoirs.

A great number of utilities have a ratio of earthquake-resistant service reservoirs of 0%, and the average across Japan in FY2005 is 20%; earthquake-proofing is thus not at a satisfactory level. However, looking at the change in the earthquake-resistance ratio from FY2005 to FY2006 in Figure 4, an improvement can be seen slightly. Furthermore, the level of earthquake-resistance for service reservoirs is higher than those of pipelines or treatment facilities, since it is easier for service reservoirs to be improved.

$$\text{Ratio of earthquake-resistant service reservoirs (\%)} = (\text{capacity of earthquake-resistant service reservoirs} / \text{capacity of all service reservoirs}) \times 100 \qquad (2)$$

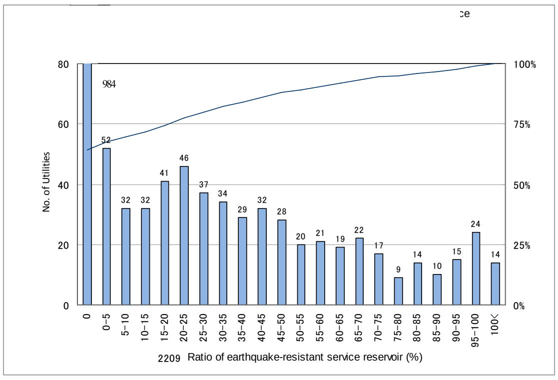


Figure 3: Ratio of earthquake-resistant service reservoirs (Number of utilities: 1,532)

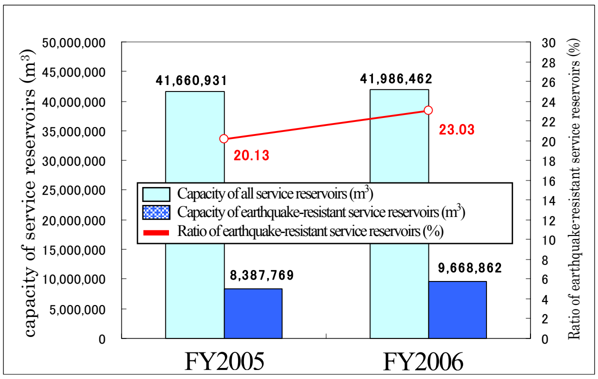


Figure 4: Comparison of the ratio of earthquake-resistant service reservoirs across two fiscal years

Ratio of Earthquake-Resistant Pipelines

The frequency histogram of the ratio of earthquake-resistant pipelines is shown in Figure 5. This is defined as the length of earthquake-resistant pipelines divided by total pipeline length.

A great number of utilities have a ratio of earthquake-resistant pipelines of less than 2%, and the average across Japan in FY2005 is 11%; earthquake-proofing is thus not at a satisfactory level. However, looking at the change in the earthquake-resistance ratio from FY2005 to FY2006 in Figure 6, an improvement can be seen slightly. In addition to the fact that trunk pipelines, like treatment facilities, are required to reach an extremely high level of earthquake-resistance (rank A against level 2 earthquakes), there are also many cases where it is difficult to replace them ; as a result, the earthquake-resistance ratio has not shown any great improvement.

$$\text{Ratio of earthquake-resistant pipelines (\%)} = (\text{length of earthquake-resistant pipelines} / \text{total pipeline length}) \times 100 \qquad (3)$$

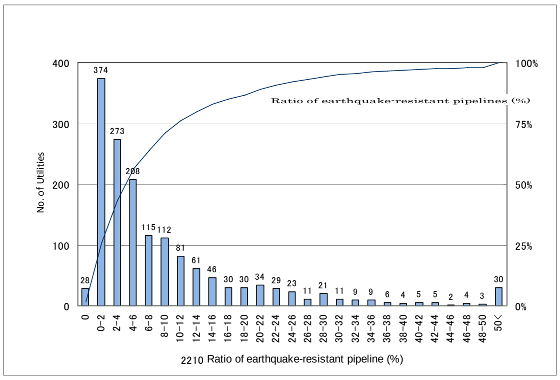


Figure 5: Ratio of earthquake-resistant pipelines
(Number of utilities: 1,564)

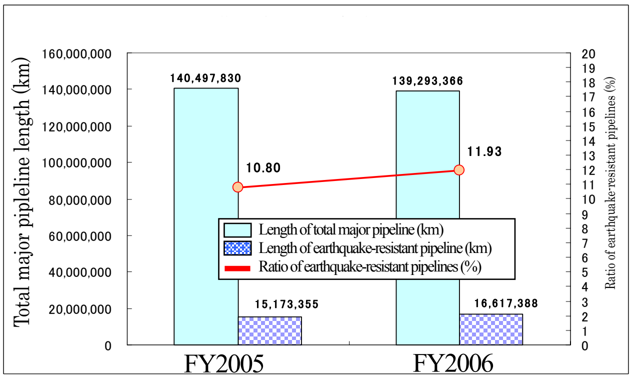


Figure 6: Comparison of the ratio of earthquake-resistant core pipelines across two fiscal years

Note: Ductile cast iron pipes with earthquake-proof joints, steel pipes with welded joints, and high-density polyethylene pipes with fused joints are considered high performance earthquake-proof pipe.

PLANNING TO UPGRADE THE EARTHQUAKE-RESISTANCE OF FACILITIES

In March 2008, Ministry of Health, Labour and Welfare (MHLW) revised “Technical Criteria of Drinking-Water Facilities” and published “Guidelines for Planning to Upgrade the Earthquake Resistance of Drinking-Water Facilities” in April 2008, in order to strengthen and accelerate earthquake-proofing [3]. Responding to the publication of the guidelines, JWRC published “Explanatory Manual of the Guidelines for Planning to Upgrade the Earthquake Resistance of Drinking-Water Facilities” in October 2008, which includes explanations and reference materials of the guidelines [4].

The workflow of formulating plans for earthquake-proofing is shown below, with the steps from (1) to (4) in order.

<The workflow of formulating plans for earthquake-proofing>

(1) Method of forecasting seismic damage to drinking-water facilities
→ A performance assessment of the earthquake-resistance of facilities in their current state and forecast of seismic damage to pipelines is carried out.

(2) Setting objectives for earthquake-proofing
→ Objectives are created for the time period of restoration, the amount of the emergency water supply, and the earthquake-resistant ratio of the trunk pipelines.

(3) Technique of earthquake-proofing
→ The techniques of the earthquake-proofing of each facility and system are selected.

(4) Method of formulating plans for earthquake-proofing
→ The most realistic and effective techniques from the menu of (3) are determined for each drinking-water utility, based on the objectives set in (2).

Forecasting seismic damage to drinking-water facilities

The Explanatory manual presents a number of methods to forecast seismic damage to drinkingwater facilities. The targets of seismic damage forecast can be categorized into two; one is structures and equipment such as treatment plants and service reservoirs, and another is pipelines such as water conveyance and water distribution pipelines. The methods of forecasting are as follows.

Structures and equipment

A simple method of performance assessment on earthquake-resistance for structures and equipment is described in the "Technical Performance Assessment Manual for Drinking Water Facilities".
More about this method will be shown later.

Pipelines

The damage estimation formula shown below is presented for pipelines. The formula has been created through the analysis of damage at the Hyougoken-nambu Earthquake in 1995. Correction factors in the formula are set due to pipe type, pipe diameter, terrain and soil, and possibility of liquefaction. The number of damage caused by an earthquake can be calculated per unit length.

$$Rm(\alpha) = C_p \times C_d \times C_g \times C_l \times R(\alpha) \qquad (4)$$

where
- $Rm(\alpha)$: Damage rate [1/km]
- C_d: Correction factor for pipe diameter (see Table 3)
- C_g: Correction factor for terrain and soil (see Table 3)
- C_l: Correction factor for liquefaction (see Table 3)
- $R(\alpha)$: Standard damage rate against maximum acceleration [1/km] $(=2.88\times10^{-6}\times(\alpha-100)^{1.97})$
- α: Maximum acceleration of earthquake movement [gal]

Table 2: Setting of correction factors

Pipe type C_p		Pipe diameter C_d		Terrain and soil C_g		Liquefaction C_l	
DIP[i]	0.3	-φ75	1.6	Modified mountainous area	1.1	None	1.0
DIP (S, NS, etc.)[ii]	0	φ100-φ150	1.0	Modified hilly area	1.5	Medium	2.0
CIP	1.0	φ200-φ450	0.8	Valley, former water area	3.2	High	2.4
SP	0.3	φ500-φ800	0.5	Alluvial flat	1.0		
VP	1.0			High-quality ground	0.4		
ACP	1.2						

[i] Excludes earthquake-proof joints. No break to earthquake-proof joints was informed, and thus they are to be studied separately.
[ii] Pipe type of S and NS are earthquake-proof pipe.

Setting objectives for earthquake-proofing

When setting objectives of earthquake-proofing, the earthquake-proofing itself should be categorized into pre-earthquake countermeasures and post-earthquake ones.

Pre-earthquake countermeasures

As facilities have their own importance according to roles, the most important facilities should be selected as the objectives of earthquake-proofing. The important facilities and the earthquake-resistance performance required are summarized below.

<Important drinkingwater facilities>

- Upstream facilities
  Water intake facilities, water storage facilities, water conveyance facilities, treatment facilities, water transmission facilities
- Core facilities in water distribution network
  Water distribution trunk mains, service reservoirs and pomp stations which are directly connected to such mains, and service reservoirs in small water supply systems etc.

<Required earthquake-resistance performance >

- Resistance against Level 1 seismic intensity
  →The facility does not lose its ability to function soundly.
- Resistance against Level 2 seismic intensity
  →Even if slight damage occurs, it rarely effects on the functions of the facility.

Post-earthquake countermeasures

Post-earthquake countermeasures are mainly emergency restoration and emergency water supply. After forecasting the interruption scale of water supply for the earthquake in question, time period of emergency restoration and the level of emergency water supply should be estimated. The scale of water supply interruption can be referred to Figure 7 which was established using the actual data in the Hyougoken-Nambu Earthquake in 1995.

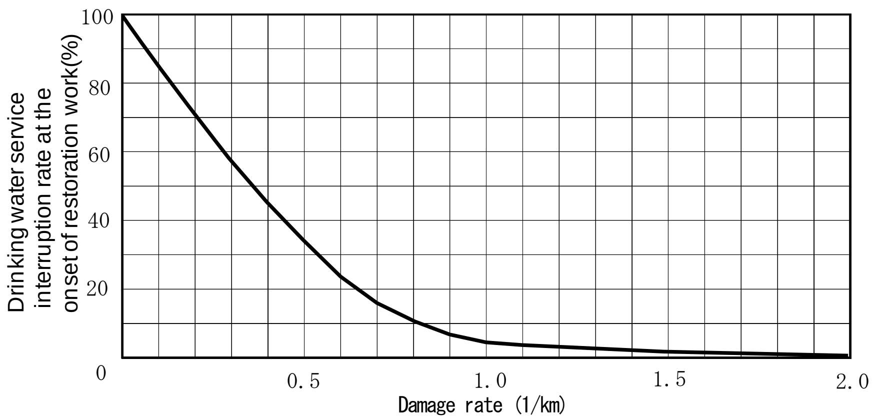


Figure7： Damage rate of water pipes and drinking water supply interruption rate at the onset of restoration work

Using this figure (or the following equations), the interruption rate at the onset of restoration work can be calculated. The rate leads to finding out the population subject to the interruption and the required amount of the emergency water supply.

Drinking water service interruption rate at the onset of restoration work

$$= 1.38 \cdot X \qquad (X < 0.50) \qquad (5)$$

$$= 1 - \frac{1}{1+18.31 \cdot X^{3.05}} \qquad (X \geqq 0.50) \qquad (6)$$

where X: Damage rate of water pipes (1/km)

A period of four weeks or less, as far as possible, is set as the objective time period of emergency restoration, taking into account of the need to lessen earthquake victims' anxiety and to create stability in their lives, as shown in the following table.

Table 3: Trends in demands for emergency water supply among earthquake victims

First 3 days	Days 4~7 (second half of first week)	Days 8~14 (second week)	Days 15~21 (third week)	Days 22~28 (fourth week)	Day 29 onwards
Minimum amount of water to live	Drinking water, Water for cooking Water for toilets	Water for laundry Bathing at evacuation area etc.	Bathing at home Laundry at home	ditto	Limit of endurance. Normal water supply level

Finally, the objective amount of water served and transportation distance of the emergency supply in respective days after the earthquake are planned. Table 4 shows an example of objective emergency water supply in Niigata City.

Table 4: Example of setting objectives for the emergency water supply in Niigata City

Stage	Stage 1	Stage 2	Stage 3
Objective amount	3 liters/person/day	20~30 liters/person/day	30~40 liters/person/day
Main purpose	Drinking water to maintain human life	Minimum water for a daily life such as cooking, laundry etc.	Securing the amount of water for a daily life
Supply method	Supply through bases Supply through transportation	Tentative water supply zones Water supply through bases/transportation	Increase in number of tentative water supply bases
Transportation distance	Within 500m from residences	ditto	Within 250m from residences

Techniques of earthquake-proofing

Techniques of earthquake-proofing can be applied to individual facilities and to the water supply system as a whole. Especially, the countermeasures should be systematically promoted to minimize the impact on earthquake victims through enhancing the backup for water distribution functions as shown below.

1) Strengthening of backup systems through connecting neighboring utilities with pipelines
2) Ensuring backup systems through connection pipes between mutual treatment plants
3) Adoption of the loop system for water transmission pipes and distribution trunk mains etc.
4) Securing backup on a systematic basis through the multiplication of facilities, the installation of bypasses, increasing the capacity of water storing facilities and multiple networks
5) Establishing a block distribution system for the water supply network where the water service area is large and where major disparities in ground levels exist, etc.

Methods of formulating plans for earthquake-proofing

The creation of plans for earthquake-proofing goes according to the following workflow. Earthquake-proofing methods are prioritized according to the earthquake-proofing objectives and the regional characteristics of the earthquakes.

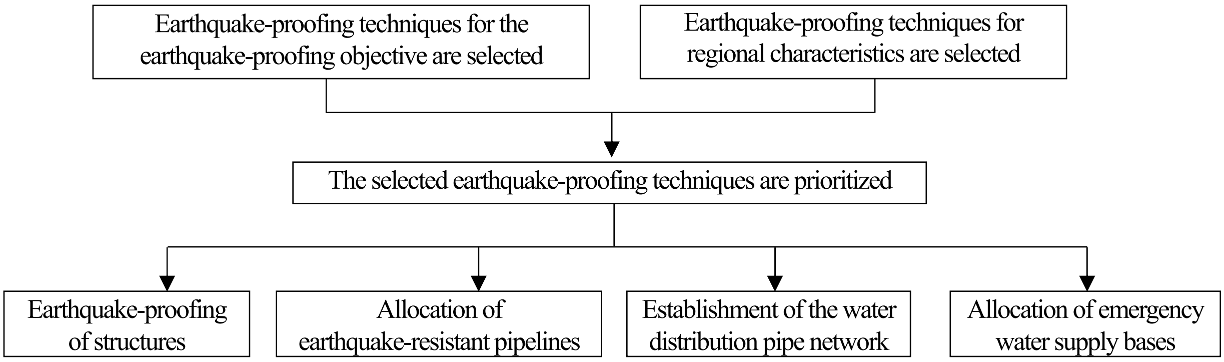


Figure8. Work flow to plan for earthquake-proofing

The selected techniques for earthquake-proofing, especially, should be applied to the following kinds of facilities.

- Facilities which have a major impact on evacuation activities and relief activities
  Examples: supply routes for medical facilities and other important facilities, water supply routes for evacuation area etc.
- Distribution pipes to facilities for vulnerable earthquake victims
  Examples: supply routes to hospitals, social welfare facilities etc.
- Facilities which are vulnerable to earthquakes due to age and luck of resistance to earthquakes
  Examples: Aging pipelines such as asbestos cement pipes
- Facilities which have a considerable impact on the vital functions of the region, the maintenance of municipal functions and restoration as fast as possible
  Examples: Core facilities of social infrastructures
- Facilities where restoration is particularly difficult
  Examples: Pipelines lying under expressways and other arterial roads, bridge-attached pipes, service reservoirs in tunnels, water channels in tunnels etc.

The following table shows examples of selected techniques for earthquake-proofing based on the relationship with earthquake-proofing objectives and regional characteristics. The effectiveness of each technique should be evaluated in terms of the objectives and the characteristics, and thus it is essential to assess its overall effectiveness.

Table 5: Examples of selected techniques for earthquake-proofing

Individual technique for earthquake-proofing	Technique for earthquake-proofing	Earthquake-proofing objective	Regional characteristics	Overall effectiveness
Reducing occurrence of seismic damage	Making water sources/facilities etc. earthquake-resistant	--	--	--
	Making pipelines earthquake-resistant	◎	○	◎
	Making piping accessory facilities earthquake-resistant	◎	○	◎
	Making water service installations earthquake-resistant	○	△	△
Minimizing impact	Block distribution system	◎	◎	○
	Loop system	◎	○	△
	Back up	◎	◎	◎
	Valve allocation	△	○	○
Accelerating restoration	Information-gathering facilities	△	○	○
	Emergency restoration work	○	○	○
Enhancing the emergency water supply	Water supply through transportation	○	○	○
	Water supply through bases	○	◎	◎

◎: Highly effective ○: Effective △: Somehow effective --: Not applicable

Finally, implementation year plans for earthquake-proofing countermeasure are formulated considering the following points.

- Annual work costs should be equalized as even as possible.
- Measures which may be effective not only as earthquake countermeasures but in other way should be implemented at an early stage.
- In the case of pipeline facilities, in order to bring maximum effective results as early as possible, the enforcement should be carried out starting from upstream and going downstream, and starting from trunk pipelines and going towards branch pipelines.
- For pipeline improvement, attention should be paid to other work, such as the replacement of aging pipelines.

SIMPLIFIED METHOD OF EVALUATING EARTHQUAKE-RESISTANCE OF WATER FACILITIES

JWRC is currently developing “The Technical Performance Assessment Manual for Drinking Water Facilities”, as part of its research project subsidized by the MHLW. This manual aims at implementing performance improvements of drinking-water facilities systematically and effectively for drinking-water utilities. It also enables the user to assess performance, including earthquake resistance, of individual equipment and device which makes up the drinking-water facilities. Further, it also shows “Check-sheet”, which is a simple and easy tool to evaluate earthquake-resistance. This report shows summary of this manual focusing on the sheet.

Outline of the Manual

The performance assessment (PA) survey may be roughly classified into five areas. Work flow is shown in Figure 9. The earthquake-resistance of each facility is evaluated using the “Check-sheet for earthquake-resistance of facilities” in “PA of Individual facility,” which is in the second area of the figure.

Evaluation Procedures	Outline of Evaluation
PA of Overall facilities ↓	Current performance level of each system is evaluated using indexes.
PA of Individual facility ↓	Current performance of each individual facility that makes up the system is evaluated.
Determination of PA results ↓ Required Performance levels being met? (Yes → End; No ↓)	After comparing required performance of the facilities with current performance, necessity of improvement is evaluated. If the required performance has been cleared, the performance assessment and evaluation ends at this step.
Selection of optimal measure(s) to improve ↓	The system/facility to be improved, the necessity, objectives, and outcome of improvement.
Implementation of improvement measures ↓ End	After the effectiveness of improvements, their consistency with requirements, and the reasonability of the improvement project are considered, the method of improvement is selected.

Figure 9. Performance Assessment (PA) Procedures

The Earthquake-Resistance Evaluation Method in Performance Assessment of Individual Facility

Performance assessment of individual facility is intended to evaluate the current performance of individual facility/equipment/device and pipeline etc. which consist of drinking-water supply facilities. Earthquake-resistance check-sheet for each individual facility/equipment is provided, enabling a simplified analysis of its earthquake-resistance.
An example of the check-sheet for a water-pipe bridge is shown in Table 2. In this check-sheet, selection of weighting-factors and multiplication of these points which are determined by the selection lead to evaluation of the earthquake resistance

Evaluation example using the check-sheet of Table 2

For Water pipe bridges made with ductile cast iron pipe, and seismic intensity scale of 6:
(Ground 1.4)×(Ground deformation 2.0)×(Foundations 1.0)×(Materials 1.4)×(Height 1.4)
×(Beam construction 2.0)×(Pipe type 1.0)×(Span 1.0)×(Shoes 1.0)×(Width of crest 0.8)
×(Joints 0.5)×(Seismic intensity scale 2.2) = 9.66.
Earthquake resistance is thus within the range 14 or less, namely evaluated as "High."

Notes:

The seismic intensity scale should be adopted taking into account the scale/magnitude of earthquake and the importance of the facilities.

Table 6. Example of Check-sheet for Earthquake-Resistance
(For water pipe bridges with ductile cast iron pipe)

Facility	Water Pipe Bridge with Ductile Cast Iron Pipe or Gray Cast Iron Pipe			
Item	Category	Weighting Factors	Points (example)	Remarks
Ground	Type I	1.0	1.4	Type I: Diluvial and rocky ground in good condition
	Type II	1.4		Type II: Diluvial and alluvial ground that does not belong to Type I or Type III
	Type III	1.2		Type III: Alluvial ground that is soft and weak
Ground deformation	No	1.0	2.0	Effect on bridge foundations due to ground deformation and slope failure caused by ground liquefaction
	Possible	2.0		
	Yes	3.0		
Foundations	With piles	1.0	1.0	
	No piles, pile bend	2.0		
Materials for bridge abutment and supports	Bricks, plain concrete	1.0	1.4	
	Other than those above	1.4		
Height of bridge abutment and supports	＜5m	1.0	1.4	The height of the bridge abutment is measured from the ground, and the height of the supports is measured from the river bed.
	5～10m	1.4		
	＞10m	1.7		
Bridge structure	Beams fixed on both ends, arches, rigid frames	1.0	2.0	
	Beams fixed on one end, continuous beams	2.0		
	Simple beams	3.0		
Pipe type	Ductile cast iron pipe (DIP)	1.0	1.0	
	cast iron pipe (CIP)	2.4		
Span	1	1.0	1.0	
	≧2	1.8		
Shoes	With device to prevent bridge failure	0.6	1.0	
	Regular	1.0		
	Movable ends	1.2		
Width of crest	Wide A/S≧1	0.8	0.8	A: width of crest, S: distance of edge (S= 0.5L+20m, however, the length of the bridge, L, is less than 100m.)
	Narrow A/S<1	1.2		
Joints	Expansion, anti-slip-out mechanism type	0.5	0.5	
	Other joints	1.0		
Seismic intensity (Japanese scale)	5	1.0	2.2	Levels according to the Japan Meteorological Agency.
	6	2.2		
	7	3.6		
Earthquake resistance	High	14＞	9.66	No damage
	Medium	14 - 28		Water supply possible despite partial damage
	Low	28＜		Severe damage or water suspension

CONCLUSION

By showing the earthquake-resistance ratio of drinkingwater facilities all over Japan with PIs, it can be seen that although the current level of earthquake-resistance in Japan cannot be described as satisfactory levels, it is slowly improving. However, in addition to recognize the national level, it is also extremely important that drinking-water utilities calculate their own PIs, to get an objective idea of what position they are among Japanese drinking-water utilities. It is hoped that utilities will proactively publish and explain PIs to the Diet and to consumers in order to gain their understanding, and to ensure accountability. The publication and explanation of PIs in this manner will enable utilities to describe the current status of their management, and will be a valid method for promoting future work plans including, for example, upgrading the earthquake-resistance of facilities.

It is important not only to accomplish a stable supply of water at normal times, but also to carry out preventive maintenance to minimize accidents and malfunctions caused by disasters. For earthquake-prone Japan in particular, implementing appropriate earthquake-proofing plans is essential, and the Guideline for Planning to Upgrade the Earthquake-resistance of Drinking-Water Facilities will contribute to this.

In order to steadily resolve issues concerning the current performance of drinkingwater facilities including earthquake-resistance, it is essential to consider the required performance, and to formulate proper and concrete plans for performance improvement and implement the plans smoothly. We sincerely hope that utilities will use of the Technical Performance Assessment Manual and carry out performance assessment of drinkingwater facilities to perform a higher level of functions of the facilities.

REFERRENCE

[1]. Japan Water Works Association. January 2005. "Guidelines for the Management and Assessment of Drinking Water Supply Service (JWWA Q 100)"

[2]. Japan Water Research Center. December 2008. "Analysis of Performance Indicators (PIs) in Guidelines for the management and Assessment of Drinking Water Services in F.Y.2006"

[3]. Ministry of Health, Labour and Welfare. April 2008. "Guidelines for Planning to Upgrade the Earthquake Resistance of Drinking-Water Facilities"

[4]. Japan Water Research Center. October 2008. "Explanatory Manual of the Guidelines for Planning to Upgrade the Earthquake Resistance of Drinking-Water Facilities"

San Francisco Public Utilities Commission Water System Improvement Program and Its Seismic Requirements

Luke Cheng[1]

ABSTRACT

The Hetch Hetchy Water System was built from 1910's through 1950's. It starts from O'Shaughnessy Dam in the Yosemite National Park and end at the City of San Francisco. In 1932, a City Charter established the San Francisco Public Utilities Commission (SFPUC) to manage the water system. Currently, The SFPUC Water System provides high-quality drinking water to 2.4 million customers in San Francisco and the Bay Area.

In 2002, SFPUC launched the $4.4 billion Water System Improvement Program (WSIP) to repair, replace, and seismically upgrade the system's deteriorating pipelines, tunnels, reservoirs, pump stations, storage tanks, and dams. The program is funded by a bond measure that was approved by San Francisco voters in November 2002 and includes more than 80 projects throughout the service area – from San Francisco to Hetch Hetchy – to be completed by the end of 2014.

This paper discusses the seismic improvement program, provides overview of the program's seismic requirements, and reports the current status.

[1]Manager, Structural Design, San Francisco Public Utilities Commission, Engineering Management Bureau, 1155 Market St. 7th Floor, San Francisco, CA 94103

1. INTRODUCTION

The San Francisco Public Utilities Commission (SFPUC) manages a complex water supply system stretching from the Sierra to the City of San Francisco and featuring a complex series of reservoirs, tunnels, pipelines, and treatment systems (Figure 1). Two unique features of this system stand out: the drinking water provided is among the purest in the world; and the system for delivering that water is almost entirely gravity fed, requiring almost no fossil fuel consumption to move water from the mountains to the customers.

The SFPUC, the third largest municipal utility in California, serves 2.5 million residential, commercial, and industrial customers in the San Francisco Bay Area. Approximately one-third of delivered water goes to retail customers in San Francisco, while wholesale deliveries to 28 suburban agencies in Alameda, Santa Clara, and San Mateo counties comprise the other two-thirds of the deliveries.

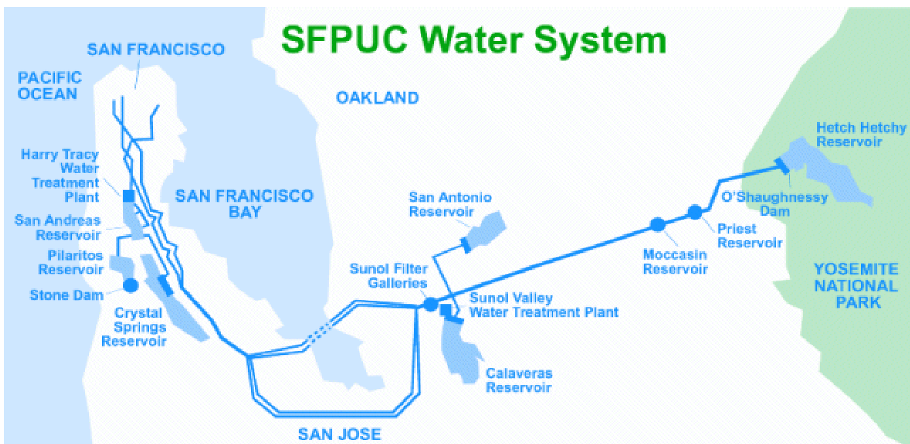


Fig. 1 SFPUC Water System

Built in the early to mid 1900's, many parts of the system are nearing the end of their working life. In addition, crucial portions of the system cross over or are near three major earthquake faults: the San Andreas fault, the Hayward fault and the Calaveras fault in the Bay Area (Figure 2). In 2002, the SFPUC, together with the San Francisco Bay area 28 wholesale customers launched a $4.4 billion Water System Improvement Program (WSIP) to repair, replace, and seismically upgrade the system's aging pipelines, tunnels, reservoirs, pump stations, storage tanks, and dams. The program will deliver the key goal and levels of service for seismic recovery through more than 80 San Francisco and regional projects through out the service area – from San Francisco to Hetch Hetchy - to be completed by the end of 2014. The WSIP is funded by a bond measure that was approved by San Francisco voters in November 2002 to repair, replace and seismically upgrade the Hetch Hetchy water system.

Fig. 2 Major Bay Area Faults

2. WATER SYSTEM IMPROVEMENT PROGRAM

The Water System Improvement Program (WSIP) includes more than 80 projects spanning seven counties – from Tuolumne County to downtown San Francisco. The WSIP involves a broad range of projects varying in size and complexity covering all aspects of the water system – from dams, reservoirs, pipelines, and tunnels to treatment facilities, pump stations, and water storage tanks.

The Program objectives include:

1. Improve the system to provide high-quality water that reliably meets all current and foreseeable local, State, and Federal requirements.
2. Reduce vulnerability of the water system to damage from earthquakes.
3. Increase system reliability to deliver water by providing the redundancy needed to accommodate outages.

About half of the projects in the WSIP are located within the City of San Francisco, and are referred to as *Local Projects*. The other half, referred to as *Regional Projects,* span across the Central Valley, southern Alameda and Santa Clara counties, up the Peninsula and into San Francisco. The Regional Projects are organized geographically into five regions: San Joaquin, Sunol Valley, Bay Division, Peninsula, and San Francisco (Figure 3).

Fig. 3 Water System Improvement Program

In the San Joaquin region, there are two pipeline projects and two treatment facilities upgrade projects.

In the Sunol Valley region, there are total of ten projects which include a new earth- and rockfill dam, a new tunnel, new and upgrade pipelines, pump stations rehabilitation and new standby power facilities.

In the Bay Division region, besides constructing a tunnel under the San Francisco Bay, the majority of the ten projects are constructing and rehabilitating pipelines and installing control valves and vaults.

In the Peninsula region, there are eighteen projects which consist of rehabilitating and constructing the existing and new pipelines, the treatment facilities, auxiliary structures of a dam, and improving the valve lots.

In the San Francisco region, two covered reservoirs are seismically upgraded.

In the San Francisco local program, there are total of thirty-five projects. Pipelines, pump stations, tanks and reservoirs are upgraded or added.

In addition to the regional and local projects, there are five water supply projects and three system wide projects. These projects provide (1) improvements related to water supply/drought protection and (2) enhancements to sustainability through improvements that optimize protection of the natural and human environment.

3. SEISMIC REQUIREMENTS

The seismic requirements include setting performance goals, assigning a performance class for each facility considered, defining design earthquakes to be used, and providing design criteria for different type of structures.

In order to reduce vulnerability of the water system from earthquakes damage to an acceptable level, performance goals were required and defined in the early stage of the Program. Also, each facility is assigned a performance class based on the criticality of the facilities. The performance goals and classes are discussed in Section 4.

Because the SPPUC facilities are located in a highly seismic active area, it is required to perform detailed geological and geotechnical studies for critical structures per the California Building Code. Seismic hazards such as fault ruptures, ground motions, liquefactions and landslides need to be evaluated. Detailed discussions are included in Section 5.

Each facility needs to be designed or upgraded to resist a prescribed earthquake. Because the SFPUC facilities are located in different areas and may subject to different earthquake motions, design earthquake for each facility has to be defined. A consistent approach to develop design earthquakes is very important. Section 6 describes the methods recommended for the WSIP structures.

The SFPUC water system consists of pipelines, tunnels, covered reservoirs, pump stations, treatment plants, administration/office buildings, storage tanks, valve houses/vaults, and dams. From structural design and rehabilitation point of view, these facilities can be classified as buildings, building-like structures, pipelines, soil retaining structures, underground structures, water retention structures, dams and reservoirs. Structures not belong to any of the above classifications will be special structures.

International Building Code (IBC) and ASCE/SEI-7 are well established code and standard used in the United States. California Building Code (CBC) has adopted IBC and ASCE/SEI-7 with some more stringent provisions. They are used to design new buildings and building-like structures, non-building structures and non-structural components. Additional standards such as ACI 350, ANSI/AWWA D100 and D110 are also used for water retention structures including tanks and covered reservoirs.

American Lifelines Alliance (ALA) Guidelines may be used for design of new pipelines and rehabilitation of existing pipelines with the exceptions of using a design earthquake of 975-year return period instead of 2475-year for SPC-III pipelines. Because ALA is not a code or a standard, the pipeline design engineer may choose any of acceptable methods by American Water Work Associations (AWWA), American Society of Civil Engineers (ASCE), American Society of Mechanical Engineers (ASME), American Petroleum Institute (API) or other national organizations.

Structures other than buildings, building-like structures and pipelines should be designed for appropriate static and seismic loads. Tunnels and dams will require special 2-D or 3-D soil-structure or soil-fluid-structure interaction analysis depending on the structure layout and dimensions. Dams and reservoirs in California are under the jurisdiction of the California Department of Water Resources' Division of Safety of Dams (DSOD). They should be designed following the guidelines by the DSOD and the United States Society of Dams (USSD).

For structures not mentioned in the above, project specific requirements will be developed using the most updated and acceptable procedures from the recognized experts.

A flowchart summarizing procedures for seismic design and evaluation of SFPUC WSIP facilities is shown in Figure 4.

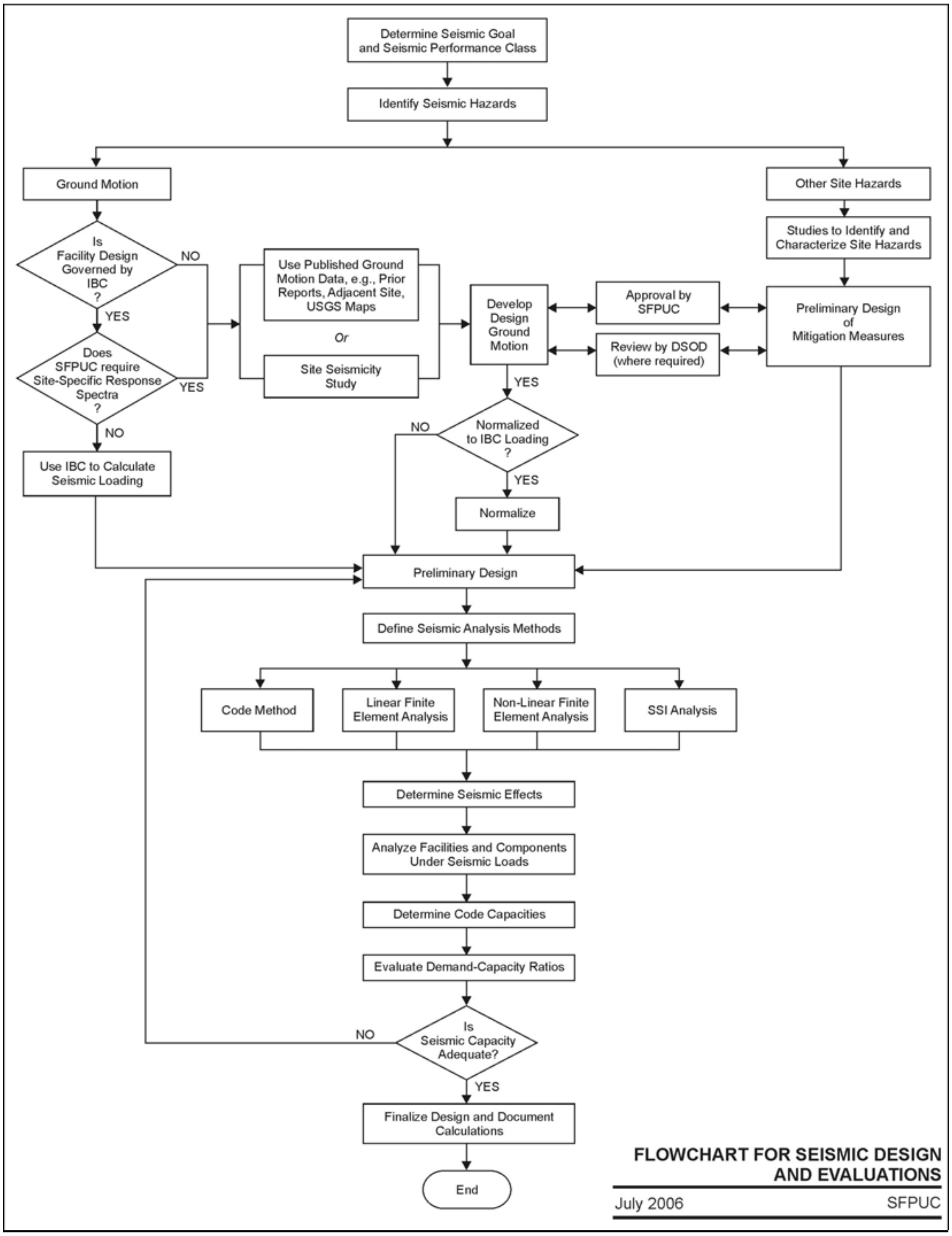


Fig. 4 Flowchart for Seismic Design and Evaluation

4. PERFORMANCE GOALS AND CLASSES

The SFPUC has committed to a basic "Level of Service Criteria" which is to be able to deliver winter day demand (WDD) of 215 million gallons per day with 24 hours after a major earthquake. This is based on the assumptions that (1) deliver WDD at least 70% of SFPUC wholesale customer's turnouts within each of the three customer groups (Santa Clara/Alameda/South San Mateo County, Northern San Mateo County, and City of San Francisco) and (2) achieve a 90% confidence level of meeting the above goal, given the occurrence of a major earthquake. To verify achievement of the service performance goals, the SFPUC will perform periodic remodeling of the SFPUC system using updated fragility data.

A new terminology "Seismic Performance Class" (SPC) is introduced in the SFPUC seismic criteria. Numerical numbers of I, II, and III are assigned according to the criticality of the structure. SPC with expected performance goals and examples are described in Table 1.

Table 1 Performance Goals and Seismic Performance Class

Performance Goal	Seismic Performance Class	Potential Examples [1]
Provide life safety protection for major earthquakes likely to affect the site. Facility may not be economically repairable in the event of such an event.	**Standard** **I**	• Administrative centers, repair shops, service centers and similar support facilities. Repair shops needed for post earthquake repairs may need to be in a higher Seismic Performance Class.
Provide life safety protection for earthquakes likely to affect the site. Facility may experience damage but should be capable of restoration to service within 30 days.	**Important** **II**	• Structures and components of the storage, distribution, treatment and control systems with some level of redundancy or for which failure does not result in an unacceptable service level. • Pressure zones with pumping plants, reservoir sites and the like providing redundancy and having no common-cause failure modes[2], shall have their facilities classified as Important, but should be capable of restoration to service within a specified period of time. (The required recovery time for these facilities will be determined by the project-specific requirements.)
Provide life safety protection for earthquakes likely to affect the site. In addition, provide reasonable expectation of post-earthquake	**Critical** **III**	• Structures and components of the storage, distribution, treatment and control systems with no redundancy or with redundancy having common-cause failure modes[2], and the failure of which results in an unacceptable service level. • Facilities located in pressure zones (or parts thereof) having no redundancy or redundancy

operability. Facility should be capable of restoration to a level of service consistent with adopted post-earthquake Level of Service goals within 24 hours.		with common-cause failure modes, are classified as Critical. Attention must also be given to flow limitations within the pressure zone when assessing redundancy. • Facilities needed for emergency response, such as emergency operations centers and emergency repair/response centers.

5. SEISMIC HAZARDS

The SFPUC facilities are located in an area where the level of seismicity is one of the highest in California. Figure 5 shows active faults, secondary faults and potentially active faults within the SFPUC water system. The hazards associated with such potential seismic activity include:

- Fault rupture at site traversed by faults;
- Ground motions generated by earthquakes occurring on nearby or distant faults;
- Instability of slopes at or near the site;
- Liquefaction, in saturated cohesionless soil strata underlying the site of a facility, that may lead to loss of bearing for shallow foundations, lateral support of deep foundations, settlements, lateral spreads and/or lateral flows, and buoyancy effects;
- Loss of strength in cohesive soil strata underlying a facility that may lead to comparable consequences.

Some of these hazards need to be identified and evaluated on an area or system-wide basis and some require site-specific investigations. Evaluation of fault activities and the potential for fault rupture across a facility and estimation of earthquake ground motions generated for a generic site condition (e.g., rock outcrop) need to be performed on an area or system-wide basis. The other hazards, including estimating earthquake ground motions to account for a local site condition, instability of slopes, liquefaction of saturated cohesionless soils, or loss of strength of cohesive soils require site-specific investigations.

To complete these hazard evaluations, WISP projects are required to complete a study including geologic, seismologic, and geotechnical aspects.

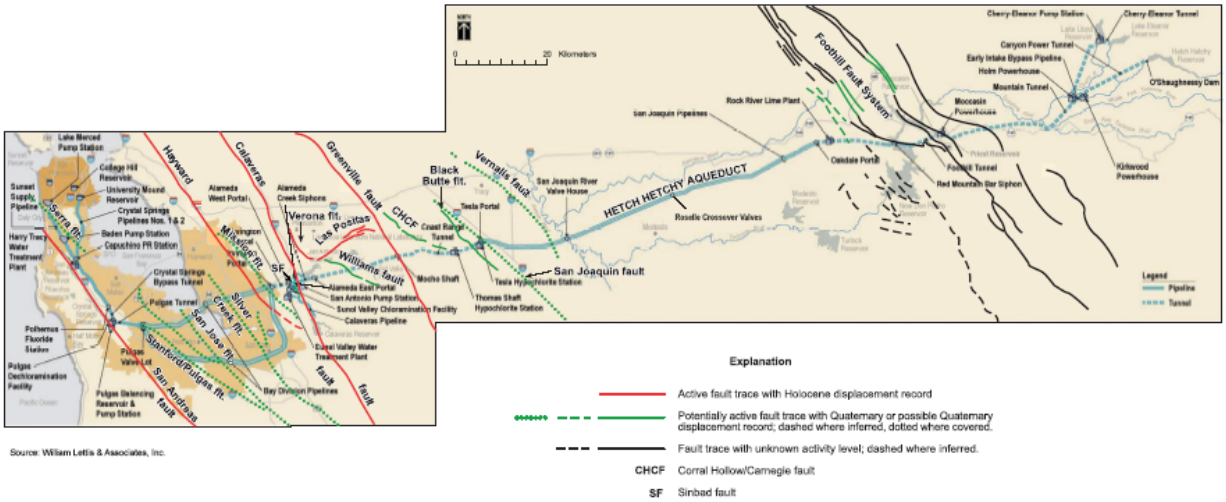


Fig. 5 Schematic Fault Map of the SFPUC Water System

6. DESIGN EARTHQUAKES

Because the SFPUC water system includes many different types of structures such as treatment plants, pump stations, vaults, valve houses, pipelines, tunnels etc., design earthquakes are different for each facility.

For the design of new structures such as buildings, building-like structures (defined as a structure which has vertical and lateral systems similar to buildings and is designed, fabricated and erected in a manner similar to buildings), tanks, vaults, treatment/filter basins, equipment anchorage, and any other structures covered in ASCE/SEI 7, the design earthquakes should be the ground motions as described in IBC which is based on a Maximum Considered Earthquake (MCE) modified with appropriate design parameters.

For the rehabilitation of existing structures, the design earthquakes should be the ground motions as defined in ASCE/SEI 41. They are Basic Safety Earthquake 1 (BSE-1) and Basic Safety Earthquake 2 (BSE-2). They can be defined on either a probabilistic or deterministic basis. The SFPUC seismic criteria define BSE-1 as an earthquake with a level of ground shaking having a 10% probability of exceedance over a 50-year interval (475-year return period earthquake) and BSE-2 as one with a level of ground shaking having a 2% probability of exceedance over a 50-year interval (2475-year return period earthquake). When the MCE maps do not adequately characterize the local hazard, site-specific procedures should be used.

For the assessment of seismic geozards and the design of pipelines and tunnels, the design earthquake ground motions should be determined by probabilistic procedures. Pipelines and tunnels in SPC I and SPC II should be designed to resist the 475-year return period earthquakes. SPC III pipelines and tunnels should use the 975-year return period earthquakes. The design earthquake need not exceed a deterministic limit taken as the 84th percentile level earthquake on San Andreas, Hayward, and Calaveras faults.

7. CURRENT REGIONAL PROJECTS STATUS

As of July 1, 2009, there are two projects in the Planning Phase, eleven projects in the Design Phase, six projects in the Bid and Award Phase, five projects in the Construction Phase, two projects in the Closed-out Phase, eight projects are completed, one project has not been initiated, and eleven projects have multiple active phases,

The relative performance progress of the different WSIP regions as of July 1, 2009 is summarized in Table 2 below.

Table 2 Regional Performance as of July 1, 2009

Region	**% Planned**	**% Actual**
San Joaquin	17.1%	16.7%
Sunol Valley	12.3%	12.0%
Bay Division	14.6%	14.8%
Peninsula	14.8%	14.8%
San Francisco	48.7%	48.5%
System-Wide	30.1%	29.0%
Regional Program Cumulative	**16.7%**	**16.6%**

8. CONCLUSIONS

The SFPUC WSIP is one of the largest water infrastructure programs in the nation and the largest infrastructure program ever undertaken by the City of San Francisco. The goal of the Program's seismic criteria is to meet the performance goal of providing water to majority of the customers in a short time after a major earthquake. The criteria for design of new structures and rehabilitation of existing facilities in the SFPUC water system improvement program are in accordance with the latest codes and standards with the supplements of most reliable updated information. As of July 1, 2009, overall, actual performance on the program is tracking very close to planned performance.

ACKNOWLEDGEMENTS

The author wishes to thank Mr. Harlan Kelly, Assistant General Manager Infrastructure, Ms. Kathy How, Engineering Bureau Manager, and Ms. Susan Yee, Deputy Engineering Bureau Manager, for their support and encouragement for the paper. The author also would like to thank the following individuals for their valuable contributions to the WSIP Seismic Requirements: Dr. Norman A. Abrahamson of Pacific Gas & Electric Co, Prof. I. M. Idriss of University of California, Davis, Prof. Jack Moehle of University of California, Berkeley, Prof. Thomas D. O'Rourke of Cornell University, and Ms. Simin Naaseh of Forell/Elsesser Engineers, Mr. John Eidinger of G&E Engineering Systems, Mr. Martin Czarnecki and Mr. Steven Brokken of URS Corporation, and Mr. Brian Sadden.

REFERENCES

[1]. ACI Committee 350, (2006), *Code requirements for Environmental Engineering Concrete Structures and Commentary (ACI 350 -06)*, American Concrete Institute, Farmington Hills, MI

[2]. ALA, (2005), *Seismic Guidelines for Water Pipelines*, American Lifelines Alliance

[3]. ASCE, (2005), Minimum Design *Loads for Buildings and Other Structures, ASCE/SEI 7-05*, American Society of Civil Engineers, Reston, VA

[4]. ASCE, (2003), *Seismic Evaluation of Existing Buildings, ASCE/SEI 31-03*, American Society of Civil Engineers, Reston, VA

[5]. ASCE, (2007), Seismic *Rehabilitation of Existing Buildings, ASCE/SEI 41-05*, American Society of Civil Engineers, Reston, VA

[6]. AWWA, (2004), *Wire- and Strand-Wound Circular, Prestressed Concrete Water Tanks, ANSI/AWWA D110-04*, American Water Works Association.

[7]. AWWA, (1996), *Welded Steel Water Tanks – Welded Steel Tanks for Water Storage, ANSI/AWWA D100-96,* American Water Works Association.

[8]. AWWA, (1997), *Bolted Steel Tanks – Factory-Coated Bolted Steel Tanks for Water Storage, ANSI/AWWA D103-97,* American Water Works Association.

[9]. Hashash, Y.M.A., J.J. Hook, B. Schmidt, and J.I.-C. Yao, (2001), "Seismic design and analysis of underground structure", *Tunneling and Underground Space Technology*, 2001, 16: 247-293.

[10]. ICC, (2006), *International Building Code*, International Code Council, Country Club Hills, IL

[11]. Ostadan and Penzien, (2001), "Seismic Design of Cut-and-Cover Sections of the Bay Area Rapid Transit Extension to San Francisco, Airport," 2nd U.S.-Japan SSI Workshop on Soil-Structure Interaction, Tsukuba, Japan.

[12]. Penzien, J., (2000), "Seismically Induced Racking of Tunnel Linings," *Earthquake Engineering and Structural Dynamics*, Vol. 29, 2000.

[13]. SFPUC Engineering Management Bureau, (2008), *General Seismic Requirements for Design of New Facilities and Upgrade of Existing Facilities Rev. 1,* San Francisco Public Utilities Commission

[14]. SFPUC (2009), *Water System Improvement Program Quarterly Report Q4 FY2008|2009*, San Francisco Public Utilities Commission, August 20, 2009

[15]. USACE, (2003), *Structural Design and Evaluation of Outlet Works,* EM 1110-2-2400, U.S. Army Corps of Engineers

[16]. USSD, (1995), *Guidelines for Earthquake Design and Evaluation of Structures Appurtenant to Dams*, United States Society of Dams

[17]. USSD, (2000), *Observed Performance of Dams During Earthquakes, Volume II, USSD 2003a, Guidelines for Inspection of Dams After Earthquakes*, United States Society of Dams

[18]. USSD (2003b), *Guidelines on Design Features of Dams to Effectively Resist Seismic Ground Motion,* United States Society of Dams,

[19]. USSD (2003c), *Dam Modifications to Improve Performance During Strong Earthquakes,* United States Society of Dams,

[20]. Wang, Jaw-Nah, (1993), "Seismic Design of Tunnels—A Simple State-of-the-Art Design approach," Monograph 7, Parsons Brinckerhoff Quade & Douglas, Inc., June 1993.

Evaluation of Effects of Recent Strong Motion Records on Water Supply Facilities

Kimiyasu Ohtake, Yasushi Taguchi and Tatsuo Ohmachi

ABSTRACT

After the 1995 Kobe earthquake, seismograph networks have been deployed. Several records of peak ground surface acceleration in recent strong earthquakes exceeded the design earthquake motion (Level II earthquake motion) of the 1997 seismic design code for water supply facilities.

The purpose of this study is to evaluate effects of the recent strong motion records on water supply facilities; we analyzed the followings:

1. The design response spectrum (Level II earthquake motion) of the 1997 seismic design code has been compared with the response acceleration spectrum of strong motion observations that recorded at JMA seismic intensity 6 after the 1995 Kobe earthquake.
2. Using the recent strong motion records, response of the reinforced concrete reservoir tank has been simulated with the dynamic nonlinear analysis.

The important findings in this study are; (1) the response acceleration spectrum of recent strong motion records exceeded the design response spectrum, and (2) the tendency of the results of dynamic analysis was different depending on the soil type.

Kimiyasu Ohtake, Doctor of Engineer, Nippon Jogesuido Sekkei Co., Ltd. 7-20-9, Nishi-Gotanda Shinagawa, Tokyo Japan 141-0031

Yasushi Taguchi, Director of Engineering Department, Japan Water Works Association (JWWA), 4-8-9, Kudan-minami, Chiyoda-ku, Tokyo, 102-0074 JAPAN

Tatsuo Ohmachi, Professor of Engineering, Department of Built Environment, Tokyo Institute of Technology, 4259 Nagatsuta-cho, Midori-ku, Yokohama Japan 226-8502

INTRODUCTION

In Japan, after the 1995 Hyogoken-Nanbu (Kobe) earthquake, seismograph networks have been deployed by JMA (Japan Meteorological Agency), NIED (National research Institute for Earth science and Disaster prevention) and local governments.

At present, there are about 4200 seismometers for seismic intensity observation which has recorded strong motion, with JMA seismic intensity of more than 6. Several records of ground surface acceleration exceeded the design earthquake motion (Level II earthquake motion) of the 1997 seismic design code for water supply facilities.

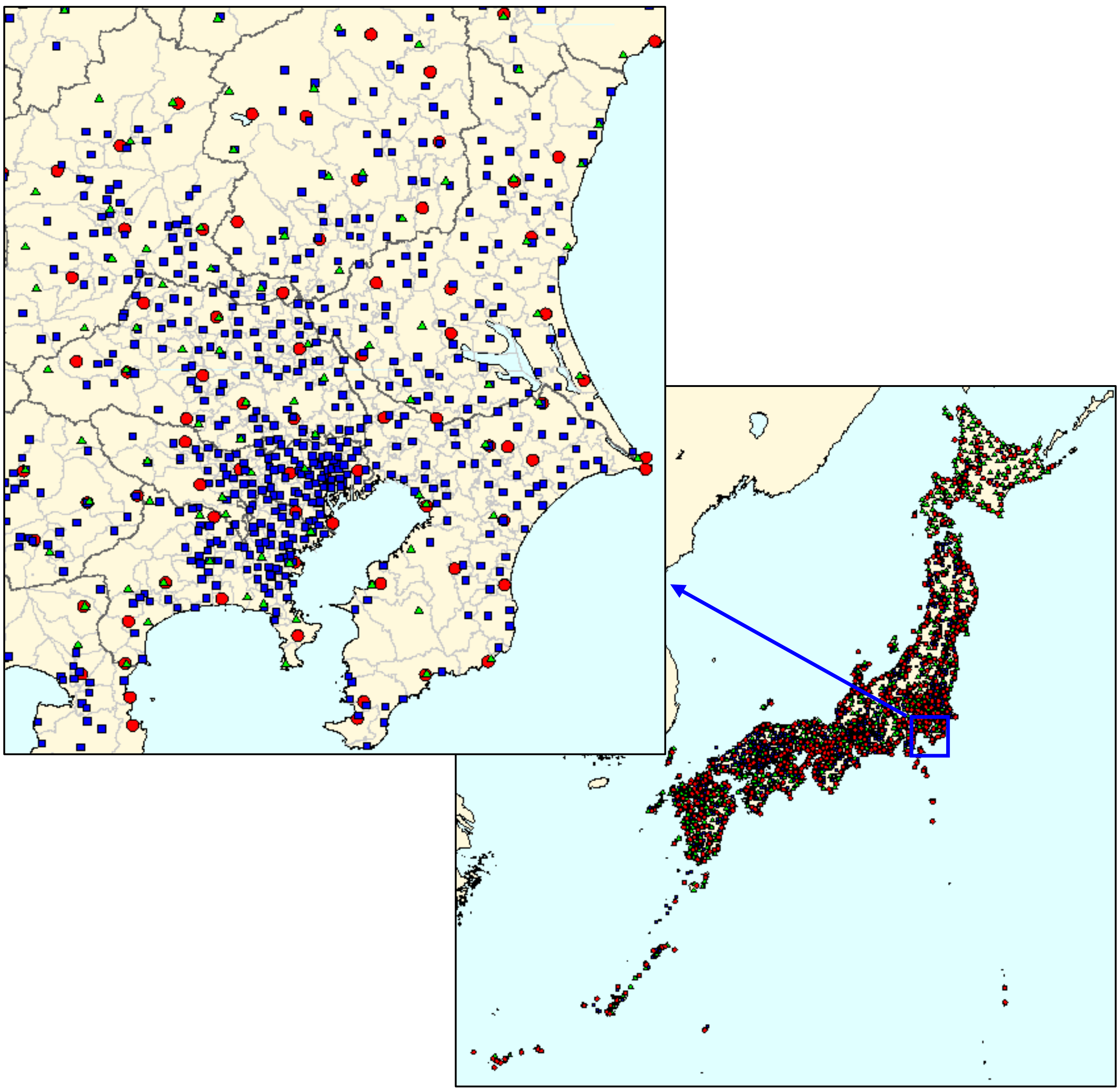


Figure1. Seismometers for seismic intensity observation in Japan[1]

COMPARISON OF THE DESIGN RESPONSE SPECTRUM WITH THE RESPONSE ACCELERATION SPECTRUM OF THE OBSERVATION

Earthquake data used in this study

Using strong motion records from inland earthquakes with JMA seismic intensity more than 6 after the 1995 Kobe earthquake, we compared the response spectrum of these data with the design response spectrum (Level II earthquake motion) of the 1997 seismic design code for water supply facilities (hereafter the 1997 seismic design code)[2].

1. The Western Tottori Prefecture Earthquake in 2000 (M_J=7.3)
2. The Mid Niigata Prefecture Earthquake in 2004 (M_J=6.8)
3. The Noto Hanto Earthquake in 2007 (M_J=6.9)
4. The Niigataken Chuetsu-oki Earthquake in 2007 (M_J=6.8)
5. The Iwate-Miyagi Nairiku Earthquake in 2008 (M_J=7.2)

Strong motion records used in this study

The strong motion records were obtained from the database of the Kyoshin Network (K-Net), the Kiban-Kyoshin Network(KiK-Net) and JMA database. K-net [3] and KiK-net [4] data maintained by NIED of Japan have online records of earthquakes occurred in all over Japan since 1996.

As shown in Table1, there were in total 92 sets of 2 components acceleration data recorded at 46 stations.

TABLE 1. Strong motion records used in this study (JMA seismic intensity more than 6)

Soil Type	Soil Name	Number of strong motion records
Soil Type 1	Bed rock, Diluvium	19 stations×2components = 38
Soil Type 2	Alluvium	15 stations×2components = 30
Soil Type 3	Soft soil	12 stations×2components = 24
	Total	46 stations×2components = 92

Results of the calculations of the response acceleration spectrum

Figure 2 shows the response acceleration spectrum of strong motion records at soil type 1 (damping factor 5%). The blue line is a non-exceedance probability value of 90% for the response acceleration spectrum of strong motion records. The same calculations were conducted for soil types 2and 3, with the results shown in Figure 3.

Figure 4 shows the design response spectrum (Level II earthquake motion) from the 1997 seismic design code. The design response spectrum was set based on the strong motion observations recorded during the 1995 Kobe earthquake.

Figures 3 and 4 indicated that the 90% non-exceedance probability value for the response acceleration spectrum of the strong motion records is larger than the design response spectrum of the 1997 seismic design code. In particular, its maximum for soil type 1 is more than three times larger than the design response spectrum.

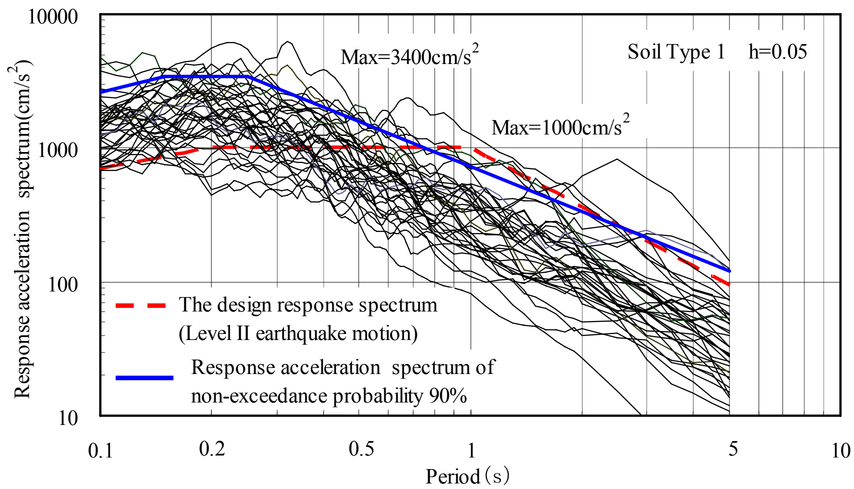


Figure2. Response acceleration spectrum of strong motions recorded at JMA seismic intensity more than 6 after the 1995 Kobe earthquake (Soil Type 1, Damping factor 5%)

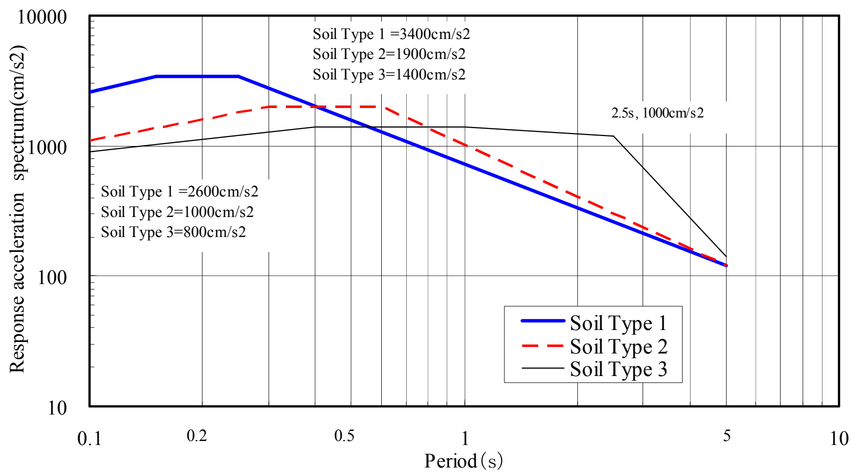


Figure 3 Response acceleration spectrum of non-exceedance probability value of 90% from the response acceleration spectrum of strong motion records

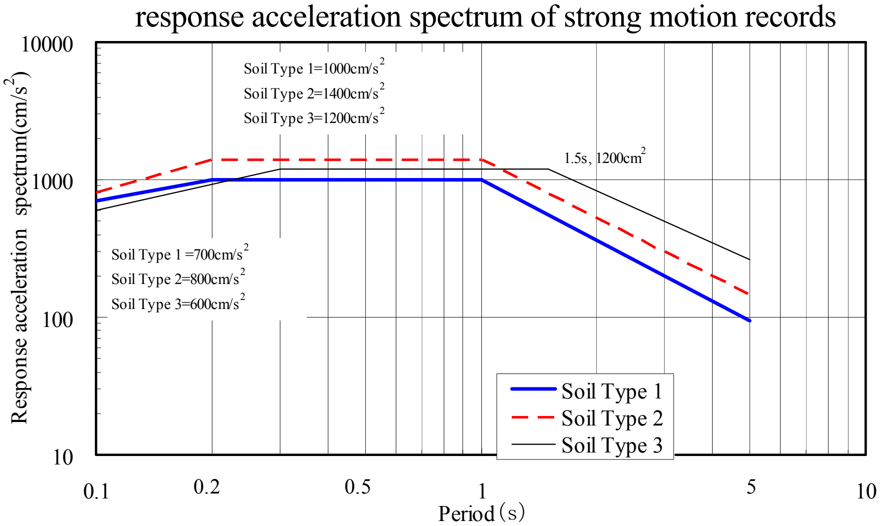


Figure 4 Design response spectrum (Level II earthquake motion) of the 1997 seismic design code

EFFECTS OF LEVELII EARTHQUAKE MOTION ON RESERVOIR TANKS USING DYNAMIC ANALYSIS

In order to evaluate the influence of recent strong motion records on water supply facilities, response of the reinforced concrete reservoir tank has been simulated with the dynamic nonlinear analysis, using the recent strong motion records and the accelerogram consistent with the design response spectrum.

Analysis conditions

Analysis cases

TABLE 2 shows the analysis cases. The soil conditions for the analysis cases were assumed to be soil types 1 and 2, with the input seismic motions of 2 and 3 types respectively.

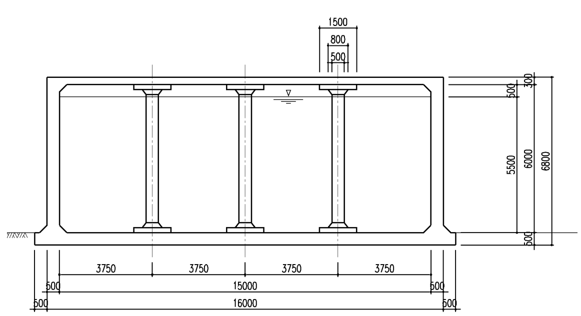
TABLE 2. CASE OF THE DYNAMIC NONLINEAR ANALYSIS

No.	Soil type	Input seismic motion	Max.ACC (cm/s^2)
I -1	Soil type1	Accelerogram consistent with design response spectrum (Level II ,Soil type 1)	538
I -2	Soil type 1	K-net Tokamachi-NS (Observation record during the Mid Niigata Prefecture Earthquake)	1,715
II -1	Soil type 2	Accelerogram consistent with design response spectrum (Level II ,Soil type 2)	680
II -2	Soil type 2	JMA Kawaguchimachi-EW (Observation record during the Mid Niigata Prefecture Earthquake)	1,676
II -3	Soil type 2	Takatori Station –NS (Observation record during the Kobe Earthquake)	604

Construction conditions

An overview of the structures used for the study is shown in Figure 5.

The material specifications used in the analysis are shown in TABLE 3 and the element conditions are shown in TABLE 4.



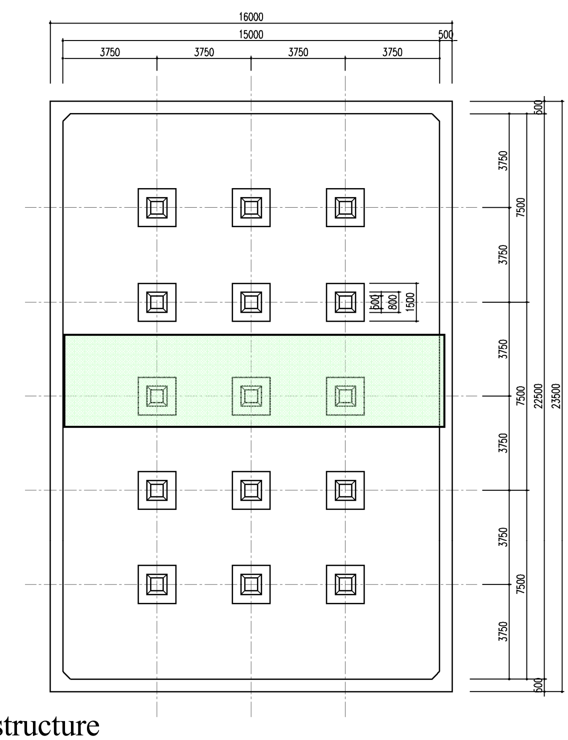


Figure 5 Section of the structure

TABLE 3 MATERIAL STRENGTH

The value of compressive strength of concrete	$24N/mm^2$
The value of tensile yield strength of reinforcing bar	$300N/mm^2$

TABLE 4 ELEMENT CONDITIONS

Element		Base	Slab	Wall	Column
Thickness(mm)		t=500	t=300	t=500	500×500
Axial reinforcing bar	Diameter and Pitch(Inside)	D16@300	D16@300	D22 @ 300	4-D19
	Diameter and Pitch(Outside)	D16@300	D16@300	D22 @ 300	4-D19

Analysis method and model

The analysis was non-linear dynamic analysis using the direct integration method. Numerical integration was carried out using the Newmark-β method (β=0.25). A time interval for the integration is set at 0.001sec.

The analysis model was a two-dimensional vertical cross-section of the reservoir tank, as shown in Figure 6. The soil surface was modeled as a plane strain element under the construction.

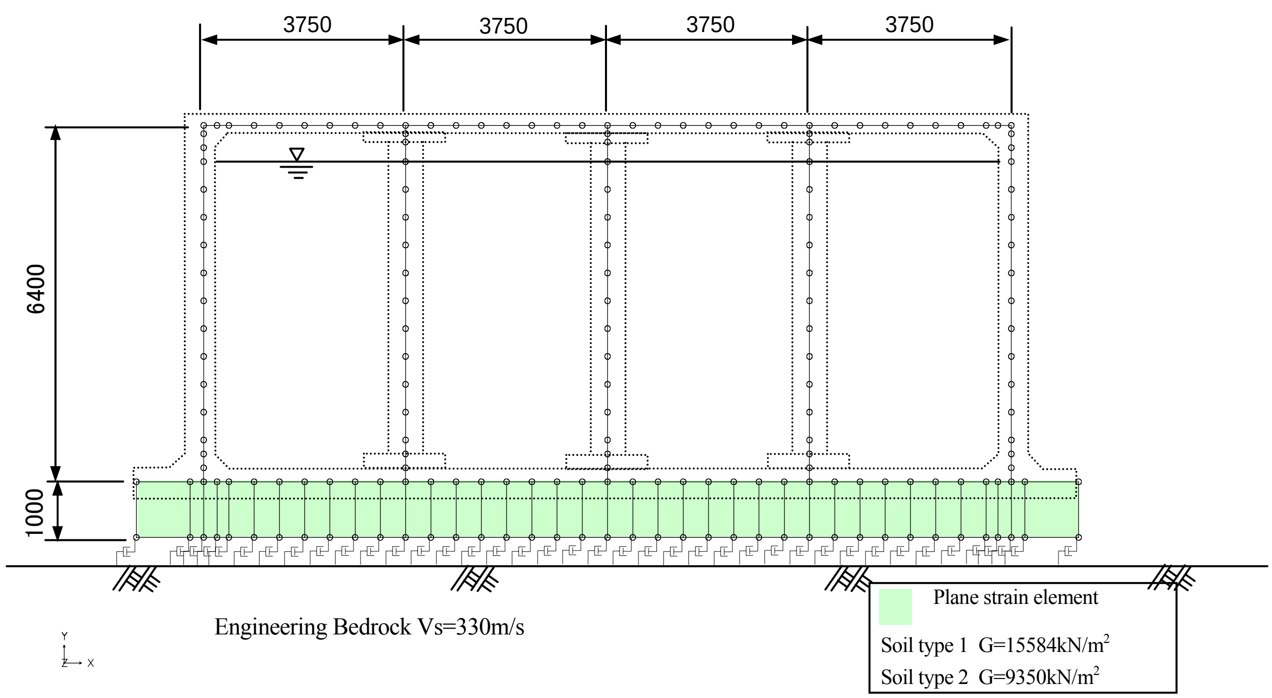


Figure6 Model of the dynamic analysis

Seismic performance

In this analysis, the response of structure should satisfy the seismic performance II. The relationship between seismic performance and resistance status of the element is shown in Figure 7.

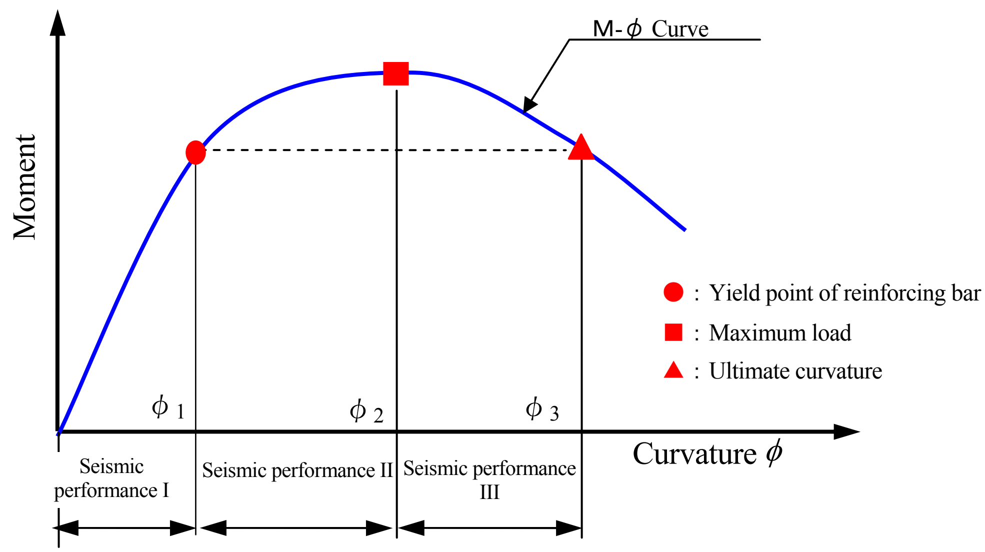


Figure 7 Relationship between the seismic performance and the M-φ curve

Evaluation of safety

Evaluation of safety was calculated using the following formula for response curvature of elements.

$$\gamma_i \cdot \frac{S_d}{R_d} \leq 1.0$$

$$S_d = \gamma_a \cdot S$$

$$R_d = \frac{R}{\gamma_b}$$

where,

S_d : Design response

R_d : Design resistance

S : Response curvature (φ)

R : Resistance curvature (φ_2)

γ_i : Structural factor (=1.0)

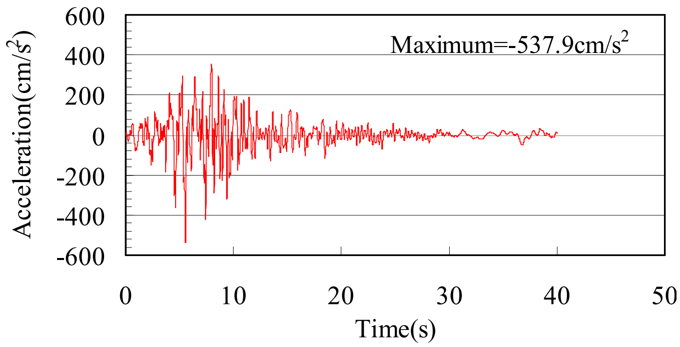
γ_a : Structural analyzing factor (=1.0)

γ_b : Material factor (=1.0)

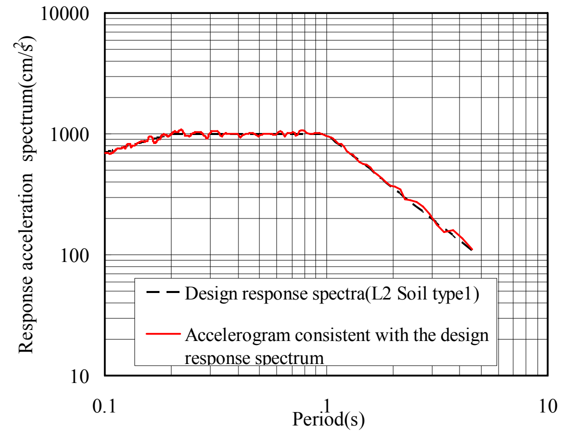
For soil type 1 model

Input seismic motion

Figure8 and Figure 9 show the input seismic motion used for the model on soil type 1. These seismic motions were input at the bottom of analysis model.

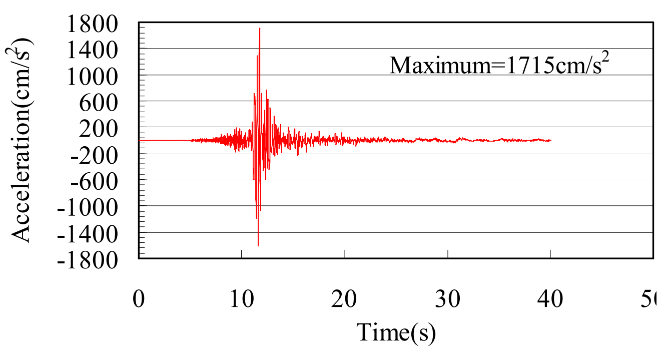


Acceleration time history

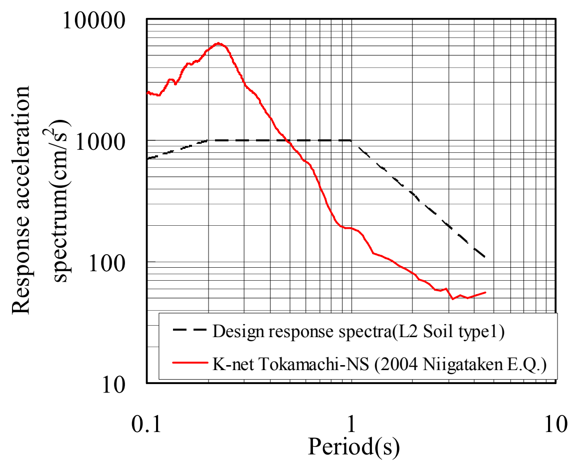


Response acceleration spectrum

Figure8 Accelerogram consistent with the design response spectrum for Soil type 1



Acceleration time history



Response acceleration spectrum

Figure9. K-net Tokamachi-NS

Analysis results

Figure 10 shows the S_d/R_d (response curvature/curvature of maximum load) for each component. For both types of input seismic motion, the S_d/R_d shows a maximum at the column, and a minimum at the base.

For Tokamachi-NS record, the results of the calculations at column indicated that the S_d/R_d was 40% smaller than other cases.

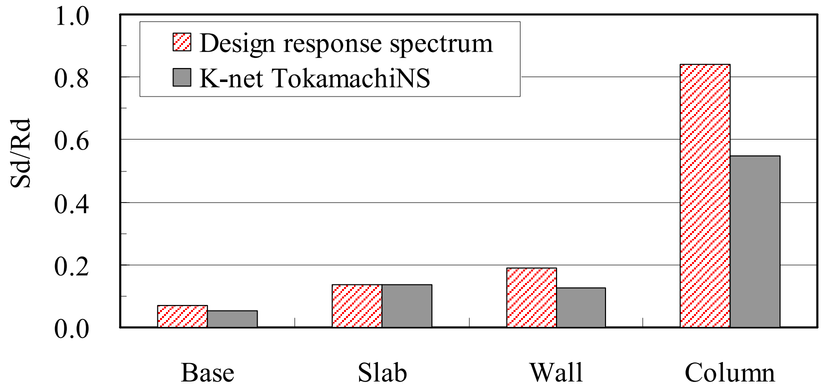
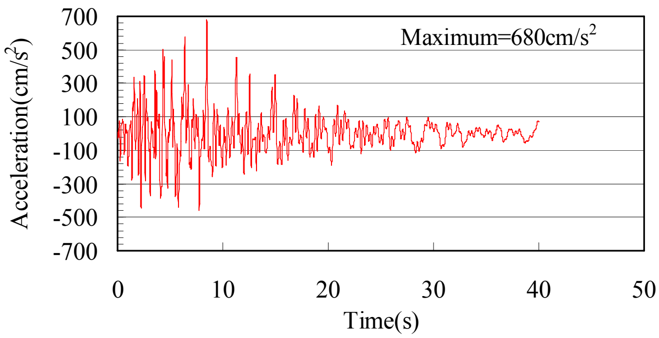


Figure10 Calculation results of Soil type 1

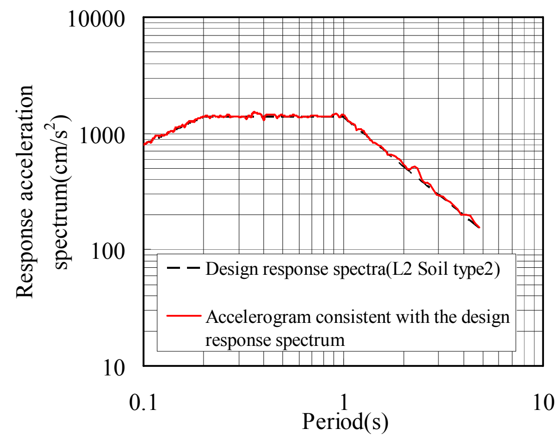
For of soil type 2 model

Input seismic motion

Figure11, Figure12 and Figure 13 show the input seismic motion used for the model on soil type 2.

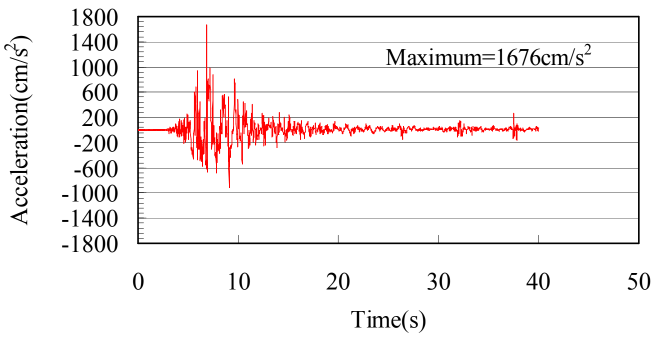


Acceleration time history

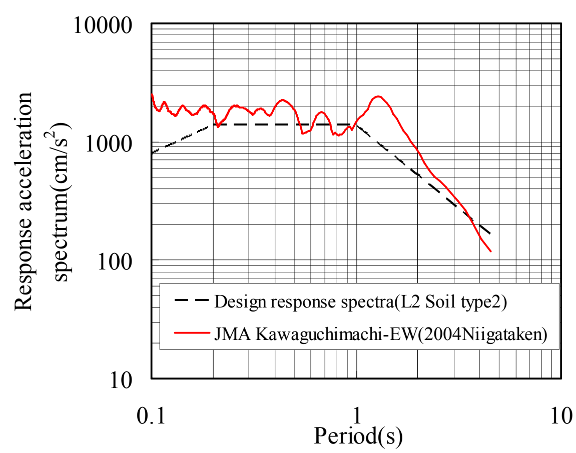


Response acceleration spectrum

Figure 11 Accelerogram consistent with the design response spectrum for Soil type 2

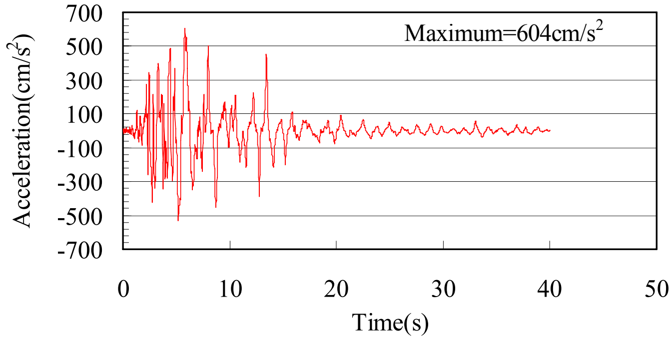


Acceleration time history

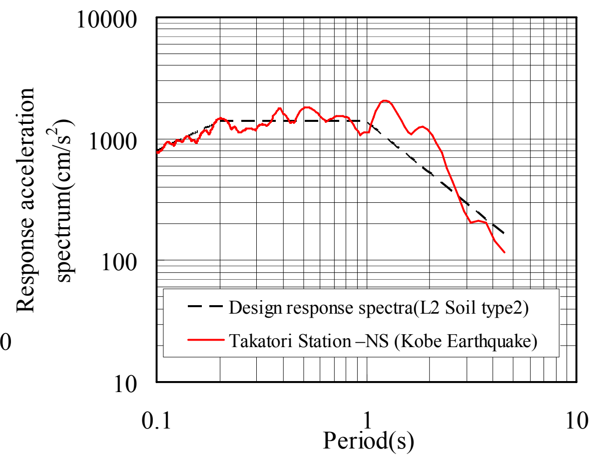


Response acceleration spectrum

Figure 12 JMA Kawaguchimachi-EW



Acceleration time history



Response acceleration spectrum

Figure 13 Takatori Station –NS

Analysis results

Figure 14 shows the S_d/R_d (response curvature/resistant curvature) for each element. For all types of seismic motion, the S_d/R_d shows a maximum at the column, and a minimum at the base.

The maximum Sd/Rd values for each element were calculated with the Kawaguchimachi EW seismic motion. For the Kawaguchi EW and Takatori NS seismic motions, the Sd/Rd was larger than 1.0 at the column, and indicated that the seismic performance 2 could not be ensured.

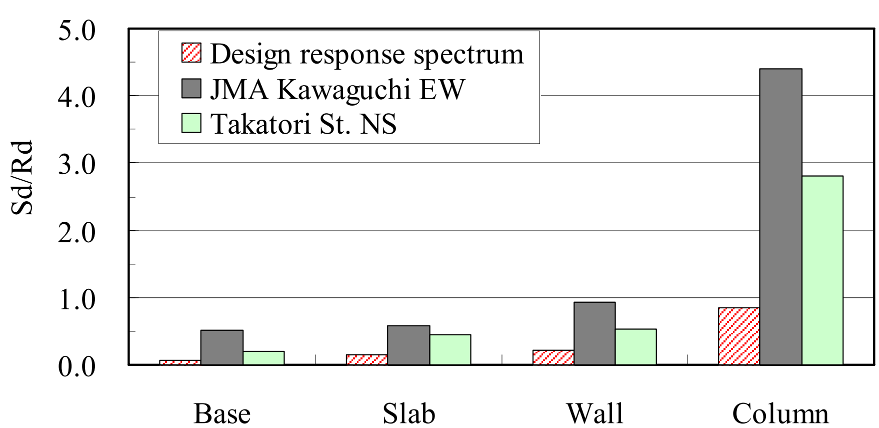


Figure 14 Calculation results of Soil type 2

OVERVIEW OF RESULTS

The results of the study are compiled and shown in TABLE 5. The response acceleration spectrum from strong motion records in recent years exceeded the design response spectrum, in particular for soil type 1 recording was more than three times larger than the 1997 design response acceleration spectrum.

On the other hand, in tests on reservoir tanks using non-linear dynamic analysis for soil type 1, the response value using accelerogram consistent with design response spectrum was slightly larger than results from accelerogram of the strong motion records. However, for soil type 2 models, the results were different from soil type 1.

TABLE 5 COMPILATION OF RESULTS

Soil type	Comparison of response spectrum	Results of the calculations for reservoir tank using non-linear dynamic analysis
Soil type 1	The response spectrum of strong motion records was more than three times lager than the design response spectrum.	• Both cases (the accelerogram consistent with the design response spectrum, and the strong motion records) ensured seismic performance II. • In case of using accelerogram consistent with design response spectrum, the response value was slightly larger than other case.
Soil type 2	The response spectrum of strong motion records was approximately 1.3 times lager than the design response spectrum.	• In cases of using the strong motion records, the results could not ensure seismic performance II (column). • In case of using accelerogram consistent with design response spectrum, the response value was smaller than other cases.
Soil type 3	The response spectrum of strong motion records was approximately 1.1 times lager than the design response spectrum.*1	—

*1 For period of 2.5 sec, the difference rises to become approximately two times larger. However, in this analysis model the characteristic period was short and as it can be considered that the influence of long period component in input seismic motions was not large, tests in soil type 3 using dynamic analysis were not calculated.

CONCLUSIONS

In this study, we evaluated the effects of the recent strong motion records on water supply facilities. The important findings in this study are; (1) the response acceleration spectrum of recent strong motion records exceeded the design response spectrum, and (2) the tendency of the results of dynamic analysis was different depending on the soil type.

On the other hand, the design response spectrum (Level II earthquake motion) in the 1997 seismic design code was established based on the damage status to water supply facilities in the 1995 Kobe earthquake. Since then there has been no significant damage due to earthquake motions to water supply facilities that have been established using the design response spectrum. In addition the evaluation using dynamic non-linear analysis were conducted on above-ground reservoir tanks, and other types of construction have yet to be conducted in this study.

Accordingly, a revision to the design response spectrum in the 1997 seismic design code should be closely checked. In the future it will be necessary to continue studies on the design response spectrum, paying full attention to the following issues:

1. Effects of construction type and status on response characteristics.
2. Evaluation of seismic performance of real structures, in term of experienced strong motion records.
3. Evaluation of effects of significant long period components on water supply facilities.

REFERENCES

[1] Japan Meteorological Agency :http://www.jma.go.jp/jma/indexe.html
[2] Japan Water Works Association: Guidelines for seismic design of water supply facilities, 2009. (in Japanese)
[3] National Research Institute for Earth Science and Disaster Prevention:K-net,http://www.k-net.bosai.go.jp
[4] National Research Institute for Earth Science and Disaster Prevention:KiK-net,http://www.kik-net.bosai.go.jp

Study on the Response of Joint of Large Diameter Pipes Caused by Earthquake

Jerry Jwo-Ran Chen[1]   Y Chuan Chou[2]

ABSTRACT

Study on the dynamic response behaviour of the connection between two piles is mostly based on the laboratory test. However, the laboratory test is expensive and is sometimes not able to obtain the desired data since the monitoring gauge is difficult to be installed at the right place from time to time. Numerical method (FEM) can have any result of desired place in the connection. However, numerical method mostly ignores the interaction between soil and connection joint because it will be very complicated if the soil and connection joint are included simultaneously in a numerical analysis. This article will compare the dynamical response between ordinary joint and flexible joint when earthquake shaking. The interaction between soil and joint structure is also taken into account in this paper. It is expected that the concept and calculation described in this paper would be helpful for the engineer in gaining the ability of analysis so that the more effective design can be developed for such problem in the future.

INTRUDUCTION

The inspections of tunnels or large pipes in past projects of our firm have indicated that cracks or leakages often take place at the connection joints of tunnels or pipes. It costs a lot to rehabilitate the connection joint damages after lifelines running for a long time. Many causes may result in the damages at the connection joints. However, earthquake is generally acknowledged the most cause to lead to the cracks or leakages at the connection joints possible. Consequently, the dynamic response at connection joints during earthquake will be examined in this article. The plain strain analysis is commonly used in the geotechnical engineering practice, such as embankment and tunnel excavation in which the length is very long in the longitudinal direction or in the direction perpendicular to the cross section to be calculated. Usually, the connection joint gives an irregular shape and is not a very long structure so that the plane strain analysis is inappropriate for the numerical calculation in this paper, Figure-1. To obtain reasonable results of calculations, it is unavoidable to employ a three-dimensional model for carrying out the dynamic response of connection joint caused by an earthquake shaking. The geometry established and the mesh generated for the connection joint in a three-dimensional model is so complicated that only few attempts would like to carry out this kind of numerical calculation. In this paper, a three-dimensional FEM analysis in the time domain for the dynamic response of the connection joints will be studied. In addition, the dynamic response of

[1] Senior Geotechnical Engineer, CECI Engineering Consultants, Inc., Taiwan. 20FL., No.185, Shinhai Rd., Taipei 10637, TAIWAN.
[2] Deputy Manager, CECI Engineering Consultants, Inc., Taiwan. 20FL., No.185, Shinhai Rd., Taipei 10637, TAIWAN

ordinary connection joint and flexible connection joint is to be compared to understand how the flexible connection joint will behave.

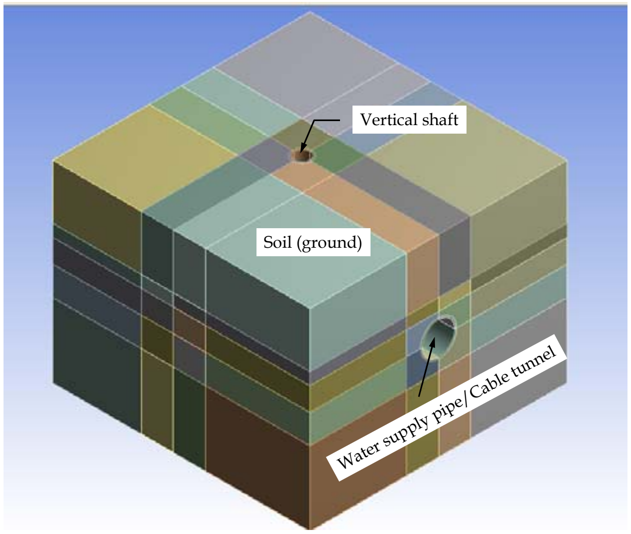


Figure 1. Layout of Geometry

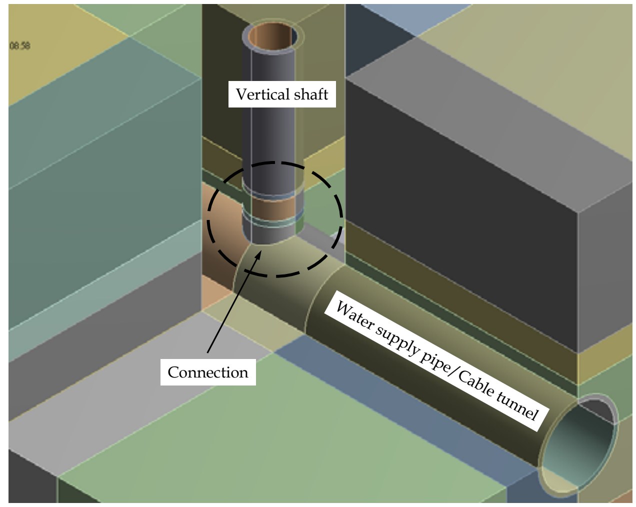


Figure 1A. Layout of Geometry

SOIL AND STRUCTURE INTERACTION

The external force derived from ground motion due to earthquake will apply on the connection joint of pipes. The deformation of joint of pipes will not have any influence on ground which is supposed to be a rigid body. Conversely, the ground is not a rigid body so that the deformation of connection joint during the earthquake would affect the ground and the ground deformation will affect the connection joints again, that is so-called soil and structure interaction. Unless the stiffness of connection joint is much less than that of the ground, the interaction between soil and ground will be generated and shall be taken into account in the numerical analysis. The ground substituted for springs is commonly adopted in most numerical analyses. The soil-structure interaction would not be generated if the ground is simply replaced by springs with no mass. In this article, soil and connection joint will be simultaneously considered in calculating the dynamical response during earthquake shaking.

ELASTIC MODULUS AND RAYLEIGH DAMPING

The performance of soil property (E_s) is static when the loading slowly apply on soil. The performance of dynamic elastic modulus (E_d) shall be taken into account if the loading exert on the soil faster. As a rule, the soil with the greater static elastic modulus also possesses a greater dynamic elastic modulus. The dynamic elastic modulus (E_d) shall be adopted in the numerical analysis when the earthquake is being attacked. The relation between static and dynamic elastic modulus is [1] :

$$\frac{E_d}{E_s} = 1 \sim 3 \tag{1}$$

It belongs to a static elastic modulus (E_s) if the evaluation is based on the undrained test (S_U) or standard penetration test blow counts (SPT N-value). The static elastic modulus (E_s) shall adequately increase so as to simulate the dynamic response of connection joint. In this paper, twice as much of elastic modulus (E_s) will be used in the numerical analysis.

The flexible connection structure and ground have to be discrete to perform the finite element analysis. The integral dynamical equilibrium equations with damping are given as following,

$$[M]\{\ddot{u}\}+[C]\{\dot{u}\}+[K]\{u\}=\{F\} \tag{2}$$

$\{\ddot{u}\}\ \{\dot{u}\}\ \{u\}$: acceleration, velocity and displacement, respectively

$\{F\}$: external force; [C] : damping matrix

Many damping theory have already been developed in which the Rayleigh damping is the most utilization possible in the numerical analysis. The Rayleigh damping matrix is defined by a linear combination of mass matrix and stiffness matrix. It gives as $[C]=\alpha[M]+\beta[K]$ (3)

α,β : the proportional constant and are defined as

$$\alpha=\xi_i\cdot\omega_o \text{ and } \beta=\xi_i/\omega_o \tag{4}$$

$$\xi_i=\frac{\alpha}{2\omega_i}+\frac{\beta\omega_i}{2} \tag{5}$$

ω_o : fundamental angular frequency

ξ_i : the damping ratio. For the saturated soil, the damping ratio is about 16%~19%.

One nature frequency of ground has to be input for calculating the α,β values in accordance the equation (4). To obtain the nature frequency of ground, a modal analysis of the integral structure, including connection joint and ground, is to be implemented. 10 dynamical modals for the integration of the ground and connection joint have been carried out. The lowest period of the first modal for the entire structure is T=0.96347, the corresponding the angular frequency can be obtained from the equation $\omega=2\pi/T$. Usually the damping ratio of soil ground is about $\xi=0.12$, then $\alpha=0.7319$ and $\beta=0.0196$ can also be obtained.

The Raylaigh damping is defined by the linear combination of mass and stiffness. Once the mass and stiffness of the entire structure are given, the Rayleigh damping will be defined as well. It indicates that the Rayleigh damping occupies no any room of computer memory in the numerical calculation that is the most advantage of Rayleigh damping employed in the finite element analysis.

BOUNDARY CONDITION

The ground domain is a semi-infinite body. The analysis domain required is cut apart from semi-infinite body. The reflection and refraction of the seismic wave will take place at the man-made cut boundary and the result of calculation may produce an impermissible error. Lysmer (1972) propose setting up a damping on the artificial boundary in which the damping will eliminate the energy of the reflection wave assembling on the boundary and then no any extra stress exert on the flexible connection joints again [2]. The artificial damping on the boundary where Lysmer proposes basically has no wave energy to reflect. Through it use, two viscous damping stresses in normal and tangential direction have been respectively applied on the boundary which are,

$$\sigma=a\rho V_P\dot{\omega} \qquad \tau=b\rho V_S\dot{u} \tag{6}$$

σ,τ : normal stress / shear stress on the damping boundary

$\dot{\omega},\dot{u}$: normal/tangential velocity component

ρ : soil density ν : poisson ratio For the body wave $a \approx b \approx 1$

V_s, V_p : S-wave velocity/P-wave velocity

$$V_S = \sqrt{\frac{G}{\rho}} \qquad V_P = \frac{V_S}{S} \qquad S = \sqrt{\frac{1-2\nu}{2(1-2\nu)}} \tag{7}$$

The spring damper of combined element-14 of ANASYS element library is adopted to exert on the boundary. The spring damper element is defined by two constant, spring constant (k) and damping coefficient (C). Usually, the coefficient of subgrade reaction of ground is around half of the elastic modulus of ground. The spring constant (k) of the combined element-14 can be obtained by employing the multiplication of subgrade reaction coefficient of ground and the dimension of area where the combined element-14 is installed.

ENGINEERING PARAMETERS FOR NUMERICAL ANALYSIS

The dynamic response of the entire structure including ground and flexible joint is much associated with the material parameters adopted in the finite element calculations. The seismic wave propagates from deep bottom to ground surface when earthquake shaking. Rayleigh damping of the entire structure is one of the most affecting factor on the dynamic response and it has been described as above. Besides, the elastic modulus of material, much related to the speed of seismic wave, is another important affecting factor on the dynamic response during earthquake. The elastic modulus of steel, concrete and rubber can be obviously gained from lots of laboratory test results. The elastic modulus of soil is much scatter in comparison with the steel. The blow counts of standard penetration test (SPT-N value) allow engineers to easily assess the elastic modulus of soil. The relation of $E = 2000 \cdot N \ (\text{kPa})$ is commonly used in evaluating the elastic modulus of sandy soil in which the N represents the SPT blow counts. Attention has to be paid that elastic modulus has to increase twice as much of elastic modulus (E_s) when the dynamic response is concerned in the numerical analysis.

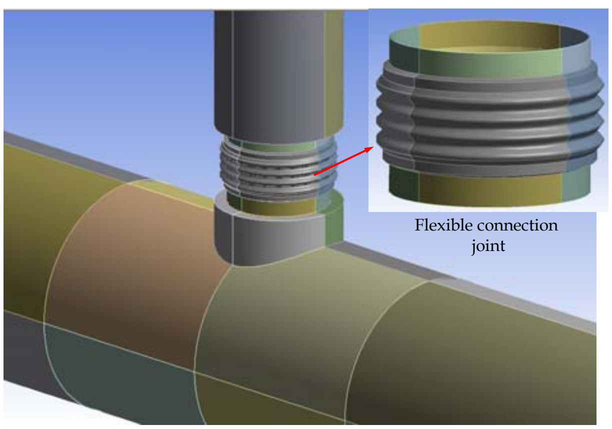


Figure 2. flexible connection joint

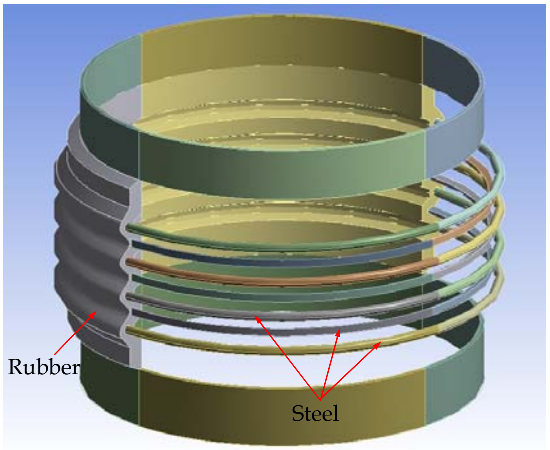


Figure 2A. Details of flexible connection joint

As a result, the engineering parameters adopted in this numerical analysis can be summarized as show in Table-I.

TABLE I. ENGINEERING PARAMETERS INPUT FOR NUMERICAL ANALYSIS

Materials	**SPT-N (Blow counts)**	**Elastic modulus (E) (kPa)**	**Poisson ratio (ν)**	**Density (kg/m^3)**
Steel	-	2.1×10^8	0.3	7850
concrete	-	2.1×10^7	0.17	2400
Rubber	-	5.0×10^4	0.48	1500
Sandy Soil	15	3.0×10^4	0.32	2000

GEOMETRY MESH FOR NUMERICAL ANALYSIS

This article aims to study on the dynamic response of flexible connection joint during the earthquake shaking. The ground has to be included in the finite element analysis since the earthquake force is able to apply on the flexible joint by through the ground medium. The ground mass plays an important role in the earthquake so that the ground cannot be simply substituted for springs with no mass in the numerical analysis. The flexible connection joint consists of mainly rubber and some steel ribs, Figure-2. The solid element-186 of ANASYS element library is used to mesh the entire structure [3]. The geometrical shape and dimension of the entire structure, including the flexible joint and the ground, are so complicate that the amount of the continue mesh is more than 200 thousands that is a huge calculation and is also very time consuming. It is near impossible to obtain the results in the transient time domain analysis. Hence, two parts, ground and pipes, have been suggested to mesh independently from one another to reduce the mesh amount, Figure-3&4. Subsequently, the contact technique has to be implemented to tie these two parts before carrying out numerical calculation. Even so, there are around 30 thousand elements and 70 thousand nodes in this FEM calculation.

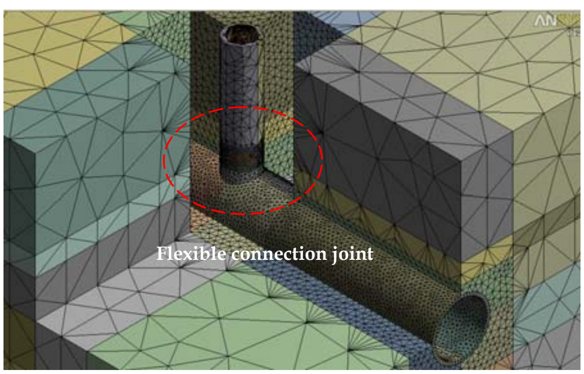


Figure 3. Mesh of flexible connection joint

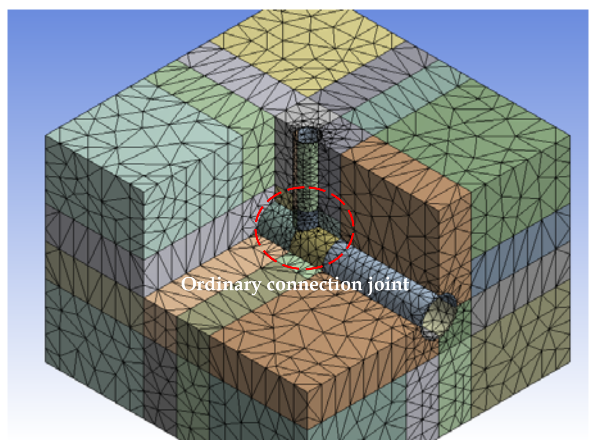


Figure 4 Mesh of ordinary connection joint

RESULTS OF NUMERICAL CALCULATIONS

The geometric shape of flexible joint including steel ribs in is too complicate to have an analytical solution. To dealing with this kind of difficulties on the engineering problems, finite difference, finite element or boundary method are commonly used to find the approximate solutions. In this article, the solution of the numerical approach for the three-dimensional transient calculation relies on the ANASYS finite element software. To understand the dynamic response during earthquake, some desired positions of connection joints have been marked to be traced, see Figure 5. The time duration of an earthquake shaking may reach 80 seconds, depending on magnitude of earthquake. The history of earthquake is in 200 record/sec. It is huge information to be calculated when carrying out the time domain transient FEM analysis. Only part of the time history record of earthquake is possible to be calculated in the FEM analysis, see Figure 6. Even so, the calculating results occupy the hard disk more than 25GB in this numerical calculation.

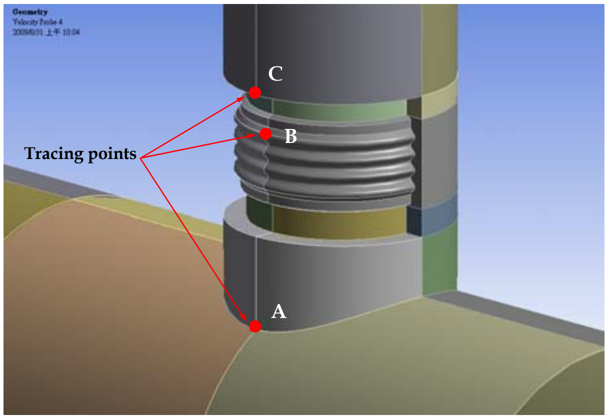


Figure 5. Tracing position

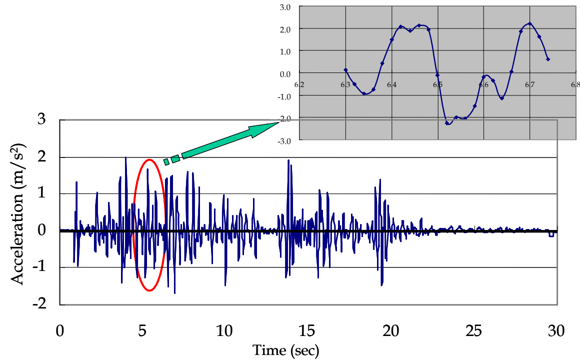


Figure 6. Earthquake history

The seismic wave propagates from deep bottom to the ground surface. Usually, the deeper ground corresponds to a stiffer ground material. As a result, the seismic wave will shake the ground in horizontal direction in accordance with the wave refraction. The seismic wave could be in any direction in which the direction of 45^o starting from the longitudinal direction of the pipe will be the most influence on the entire structure [4] and it will be taken into account in this simulation.

Figure-7&8 gives the contour of von Mises stress. It indicates that flexible connection joint shows a less stress concentration comparing to that in the ordinary connection joint. The flexible connection joint takes the advantage of reducing the stress concentration during the earthquake shaking even if it is expensive and difficult to be installed.

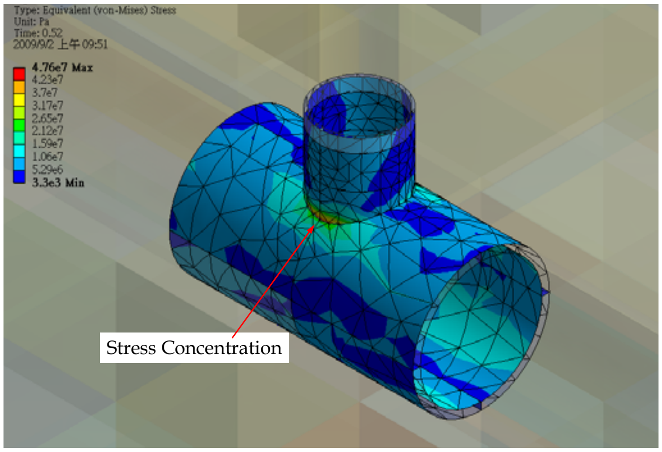


Figure 7. Stress contour of ordinary joint

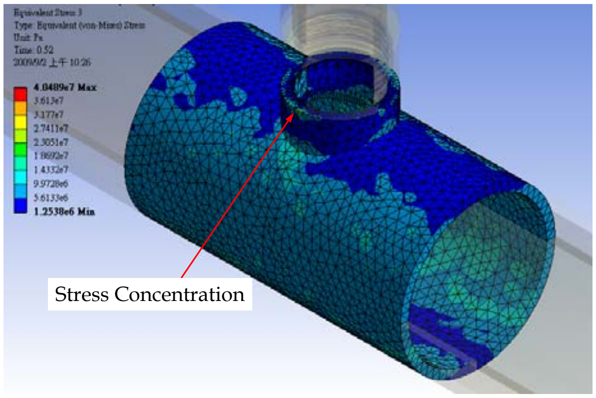


Figure 8. Stress contour of flexible joint

Figure-9&10 gives the variation of von Mises stress response at particular locations of connection joints during an earthquake shaking. In general, the seismic response in ordinary connection joint presents a greater stress than that in flexible joint. In a further examination, it can be seen that the flexible connection joint will reduce the stress concentration with regard to the ordinary connection joint as the basis of reference. It indicates that the flexible connection joint can fulfill functionality of decreasing demand for the stress variation in the earthquake occurring.

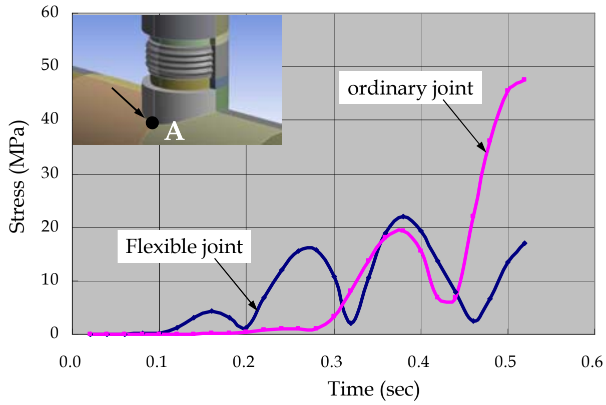


Figure 9. Response of stress history at A

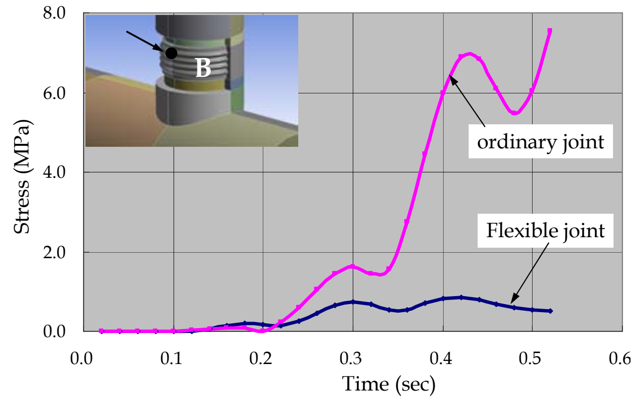


Figure 10. Response of stress history at B

The displacement response is another of use indication for expressing the stress produced in the structure as the stresses, in general, generate inversely proportional to the displacement. The variation of displacement response at particular locations of connection joints during an earthquake shaking are given in Figure-11&12. In general, the flexible connection joint presents a greater displacement response than that in ordinary connection joint. It indirectly indicates that flexible connection joint would develop less stresses in comparison with that happened in the ordinary connection joint.

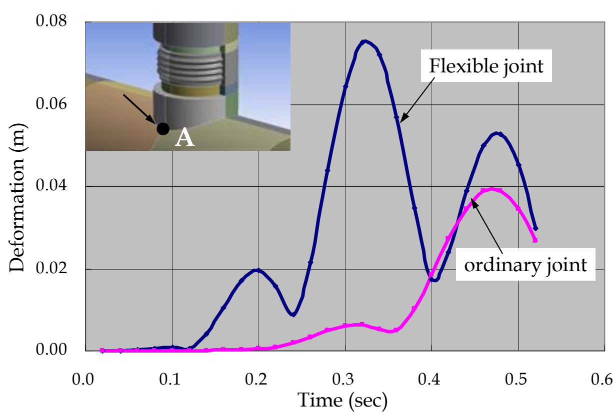


Figure 11. Response of displacement history at A

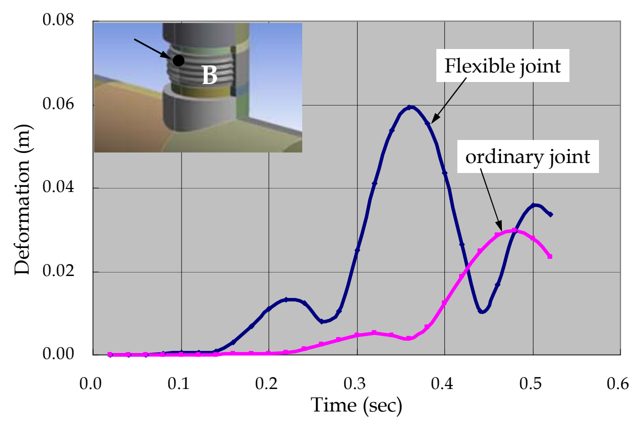


Figure 12. Response of displacement history at B

CONCLUSIONS

1. The Rayleigh damping of the entire structure is adopted in this study. The needed frequency ω for evaluating the Rayleigh damping constants α, β can be obtained by carrying out the modal analysis of the entire structure.
2. The boundary damping for no wave energy to reflect can be roughly assessed by following the Lysmer suggestions. The spring damper of combined element-14 of ANSYS element library is employed to simulate the boundary damping in which the input of spring constant (k) and damping coefficient (C) shall be adequately adjusted to meet the functionality of no energy accumulation on the boundary.

3. The stress concentration at the connection obviously takes place during earthquake shaking. The flexible connection joint results in a less stress concentration in comparison with that of which happens in the ordinary connection joint. On the contrary, the flexible connection joint will cause more displacement.
4. The flexible connection joint will reduce the stress concentration with regard to the ordinary connection joint as the basis of reference.
5. The flexible connection joint could avoid the crack and leakage during the earthquake shaking since only a less stress generation takes place.

REFERENCES

[1] *Handbook of Civil Engineering.* (1985). In Chinese, Translation from Japan..

[2] Lysmer J., & Waas G., (1972) "Shear wave in plane infinite structure", *J. EMD, ASCE, NO, EM1*

[3] *ANASYS online Manuel ..* (2009), Release 12.

[3] Chen, J.J., Chou, K.T. (2009). "Analysis Study on the Railway Moving Load Adjacent to the Diaphram Wall and Steel Supports," *J. Sino-Geotechnics.*

[4] Chen, J.J., (2009). "Inspection of Tunnel Damage of Taipei MRT," Project No.97235, Chapter 4, Final Report, Department of Geotech. Engg., CECI Engineering Consultants, Inc., Taiwan.

[5] Chen, J.J.,. (2005). "Numerical Simulation of Transform Plant Due to Earthquake," Project No.93579, Final Report, Department of Geotech. Engg., CECI Engineering Consultants, Inc., Taiwan

Development of High Seismic Performance Pipe for Crossing Active Faults

Nobuhiro Hasegawa, Toshio Imai and Nobuhisa Suzuki

ABSTRACT

Many earthquakes, such as the South Hyogo Prefecture Earthquake in 1995, occur frequently throughout Japan. Therefore, each waterworks bureau is continuously trying to improve the seismic resistance of existing pipelines. However, pipelines crossing active faults have not been actively protected even though most existing faults are clearly known, for two main reasons: the large number of faults in Japan, and the lack of a cheap protection method. There are at least 2,000 known faults in Japan, therefore many water pipelines are forced to be constructed across these faults. Fault activity causes earthquakes and displacements exceeding 1 meter. Several seismic reinforcement methods for these pipelines across faults have been suggested, such as laying pipelines in wide tunnels or installing many flexible pipes in a pipeline, but every method requires extensive investigations.

In this study, a method to increase the seismic performance of pipes across an active fault was developed. The requirements for this pipe are as follows:

1) To secure the minimum transport capacity even when a fault displacement of several meters occurs during a large earthquake
2) To ensure plastic deformation of the pipe without crack initiation or leakage
3) Easy to manufacture and construct at reasonable cost

To satisfy these conditions, the authors propose a seismic-protected pipe unit consisting of steel pipes and plural steel nodes. These nodes are able to follow large ground deformation by inelastic deformation of themselves because of their special form. These units, pipes and nodes are made of the same mild steel and have the same thickness, so it is easy to connect them to adjacent normal steel pipes at a reasonable cost.

Nobuhiro Hasegawa, Pipeline Engineer, Water Pipeline Department, JFE Engineering Corporation,
2-1 Suehiro, Tsurumi-ku, Yokohama, Japan 230-8611 (hasegawa-nobuhiro@jfe-eng.co.jp)
Toshio Imai, Manager, Water Pipeline Department, JFE Engineering Corporation,
2-1 Suehiro, Tsurumi-ku, Yokohama, Japan 230-8611 (imai-e-toshio@jfe-eng.co.jp)
Nobuhisa Suzuki, Dr. Engineering, P.E., Principal Researcher, JFE R&D Corporation,
1-1 Minami-Watarida, Kawasaki-ku, Kawasaki, Japan 210-0855 (nobuh-suzuki@jfe-steel.co.jp)

1. INTRODUCTION

1.1 Guideline for seismic design of water supply system in Japan

In 2004, "Waterworks Vision" was published by the Ministry of Health, Labour and Welfare, which set a target of increasing the percentage of earthquake-proof water supply facilities to 100% by 2013. However, there have been few efforts to improve or replace existing pipelines, and the ratio of earthquake-proof pipelines was still only 11% in 2008. Since the main cause is lack of money, it is important to develop a low-cost means of improving the seismic performance of existing pipelines.

The Japan Water Works Association (JWWA) revised its Guideline on the Seismic Design of Water Facilities in 1997, following seismic pipe failure during the South Hyogo Prefecture Earthquake. This year, the JWWA revised the guideline again to accelerate activities to improve the seismic performance of the existing water supply system. In the 1997 revision, seismic assessments of buried pipelines against Level 1 and Level 2 ground motion and lateral flow are described. In addition, the required performances of water systems for both ground motions are defined, namely, the system must not be damaged by Level 1 ground motion, and must sustain only slight damage that can be repaired immediately after a seismic event causing Level 2 ground motion.

However, the guideline does not describe the procedure of seismic design or assessment for pipelines crossing faults; it merely introduces an example of a countermeasure and the method of calculating fault displacement, even though there have been several dislocation earthquakes since the South Hyogo Prefecture Earthquake (M7.9) in 1995, including the Niigata Prefecture Chuetsu Earthquake (M7.8) in 2004, the Niigata Prefecture Chuetsu-oki Earthquake (M7.1) in 2007, and the Iwate-Miyagi Nairiku Earthquake (M7.1) in 2008.

Nevertheless, in Japan most water pipelines across faults are not protected against seismic damage.

1.2 Faults in Japan

Japan is in a zone where earthquakes occur frequently due to the activities of the four plates that surround Japan, that is, the Eurasian plate, the Pacific plate, the Philippine sea plate, and the North American plate. During the South Hyogo Prefecture Earthquake, the maximum fault displacement was 2.1 m horizontally and 1.4 m vertically. The Niigata Prefecture Chuetsu Earthquake also caused fault displacement of a similar scale. Fortunately there was no damage to water pipelines caused by fault displacement during both seismic events, because there was no pipeline crossing these faults.

However, the Kocaeli Earthquake in Turkey in 1999 caused maximum displacement of over 5 meters and serious damage to pipelines, the same as the Chi-Chi Earthquake in 1999.

Following the South Hyogo Prefecture Earthquake, the Japanese government established the Headquarters for Earthquake Research and encouraged the investigation of faults in Japan. In the last decade, over 100 main faults have been investigated and evaluated and the results recorded in a database of active faults (AIST). However, the dislocation earthquakes in recent years were caused by faults other than the main active faults. Especially, the Niigata Prefecture Chuetsu-oki Earthquake occurred at an active fault near the coast that had not been considered dangerous. Thus, there are still many active faults in the Japanese Archipelago that have not been investigated.

There are about 2,000 faults recorded in the AIST database, of which 420 are Class A active faults that have a fast displacement speed (more than 0.1 m/1,000 years). The characteristics of

Class A faults are summarized in Tables 1 to 3. The average displacement of Class A faults in Japan is 3.5 m, and half of them are classified as the reverse fault type.

Table 1. Fault displacements

Less than 1 m	1 – 3 m	3 – 5 m	More than 5 m
6	286	71	21

* Average fault displacement is 3.5 m.

Table 2. Types of faults

Normal type	Reverse type	Dextral type	Sinistral type
31	219	103	65

Table 3. Dips of faults

Less than 30°	30 – 45°	45 – 60°	More than 60°
14	167	91	145

1.3 Existing measures to protect pipelines across faults

(1) Piping with flexible joint-pipes method

Figure 1 shows an example of a technique to absorb fault displacement by laying several flexible joint-pipes. The total capacity for bending and expansion of several flexible joint-pipes absorbs the displacement. This method can be used when the pipeline is located deep underground and the location of the fault plane is clear. However, if the pipeline is laid at a shallow depth, the method costs too much because many flexible joint-pipes must be installed over a distance of about ±100 m when the faults come up to the surface ground.

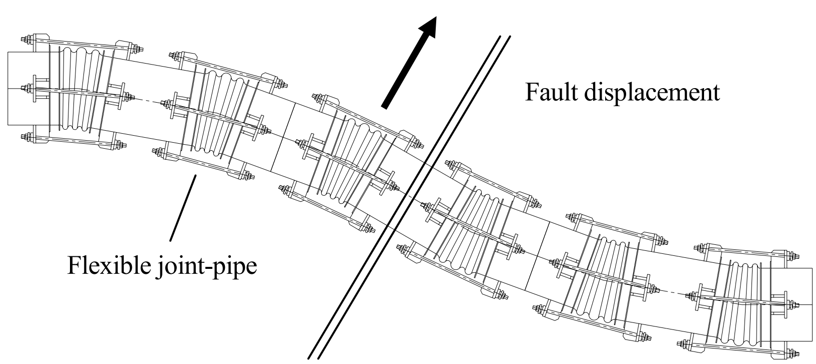


Fig. 1 Absorption of fault displacement by flexible joint-pipes

(2) Piping in tunnel method

This method aims to overcome the fault displacement by using a wide tunnel constructed to be as wide as the assumed fault displacement. The pipeline is installed in the center of the tunnel. This method also requires huge construction costs because the error of estimating the sliding plane location is ±100 m and it is necessary to construct a long tunnel with a wide cross section for piping.

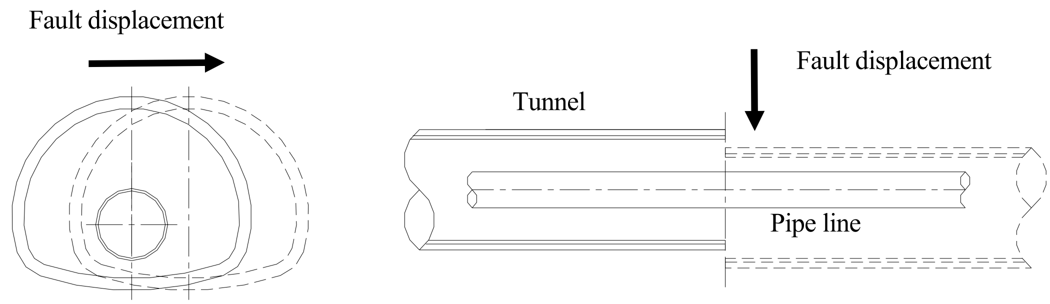


Fig. 2 Overcoming fault displacement by installing piping in a tunnel

1.4 Purpose of this study

If a large displacement with sliding of the fault occurs across an existing pipeline which is not designed to withstand fault displacement, the pipeline will collapse and water will leak. Although there are some existing methods to prevent disasters caused by fault displacement, every method incurs huge construction costs.

In this study, we developed a new seismic-protected pipe module to withstand fault displacement disasters that can be constructed with reasonable cost and by normal welding. This pipe module is made of normal mild steel (SS400) and has a special shape. The pipeline is able to withstand large displacements thanks to the plastic deformation capacity of steel.

The reasons why we selected mild steel for this pipe module were as follows:

(1) Mild steel is easy to fabricate and has superior toughness and seismic performance.
(2) High water tightness, which is essential for a water pipeline, is guaranteed by using welded joints.
(3) Mild steel has higher plastic deformation capacity than high-strength steel.

2. REQUIRED CAPACITY OF SEISMIC-PROTECTED PIPE MODULE FOR CROSSING ACTIVE FAULTS

2.1 Concept of pipe module development

If a fault displacement affects the direction of axial tension as a normal fault, then a normal steel pipeline may not collapse because the ultimate tensile strain of mild steel is over 25%.

However, it is very difficult to protect even high-strength steel pipelines from collapse, especially buckling failure, in the case of a reverse type fault that generates displacement causing axial compression and large displacement as shown in Table 1.

Therefore, we intend to protect pipelines by utilizing their flexibility, not strength, against large fault displacements. However, we decided not to adopt flexible units such as the bellows-type expansion joint which has thinner walls than the pipe unit because it may collapse under the external load when it is fully elongated. We designed the thickness of the new pipe unit to be the same or more than that of the straight pipe section.

We ignored the effects of cyclic deformations or pipe failure by low-cycle fatigue. Normally, a fault displacement can be thought of as a permanent displacement in one direction, and the return period of a big earthquake is much longer than the service period of the pipeline. Therefore, the target performances of the seismic-protected pipe module against large fault displacements are as follows:

(1) System performance (water distribution) must be maintained.
(2) Plastic deformation of the pipe or buckling is acceptable.
(3) Pipe collapse or leakage of water is not acceptable.

2.2 Method of controlling the deformation part

A normal pipeline deforms through the fault plane as shown in Fig. 3 if fault displacement occurs. It is difficult to estimate the pipe behavior such as the location and direction of deformation. Deformation of the pipeline is very complex, and compressive and tensile deformations occur alternately. As a result, the strain in the pipeline exceeds the crack initiating strain, causing water leakage or pipe collapse.

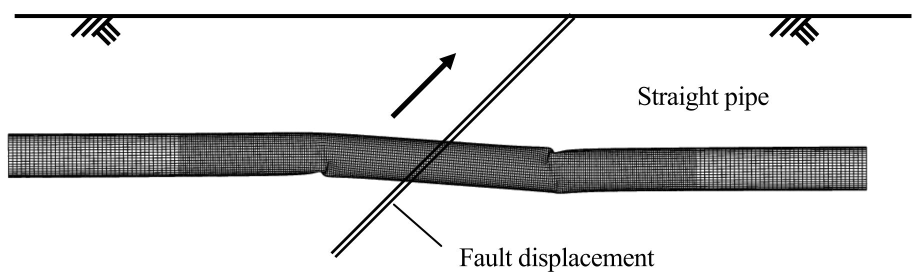


Fig. 3 Deformation of straight pipe in case of fault displacement

Therefore, we apply an initial deformation to the pipeline by setting some special pipe modules in order to control the location and direction of deformation for the fault displacement. As the deformation concentrates on these modules, other parts of the pipeline are protected from disaster.

2.3 Shape of pipe module

The buckling behavior of a cylindrical shell is greatly affected by initial imperfections, so the shape of deformation and buckling stress depend on the conditions of such imperfections. Conversely, we can control the location and direction of deformation of the pipeline by preparing an initial deformation where we want the deformation to occur. Based on this concept, we designed the shape of the seismic-protected pipe module as shown in Fig. 4. Wave width L_w of the unit is decided based on the theoretical value of the buckling half wavelength of a cylindrical shell and wave height H is decided from the thickness of the original pipe.

Wave width: $L_W = 1.72\sqrt{r \cdot t}$ (Theoretical value of half buckling wavelength) (1)

Wave height: $H = n \cdot t$ (2)

where, r: Radius of center pipe
t: Pipe thickness

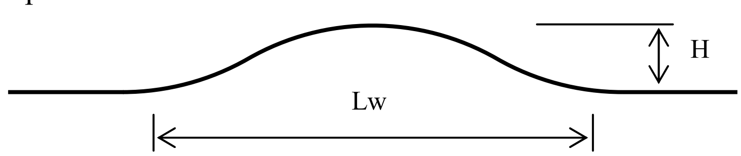


Fig. 4 Shape of pipe module (buckling wave).

2.4 Design of the optimal form of the pipe module by FEM analysis

In order to investigate the bending capability of the seismic-protected pipe module, we performed FEM analysis for the decided shape of pipe modules against bending deformation. The pipe unit is 600 mm in diameter and modeled by shell elements as shown in Fig. 5. Parametric analysis is executed for several values of wave width *L*, wave height *H* and pipe wall thickness *t* as shown in Table 4. For the case of 9-mm thickness, only *L = 1.5Lw* is executed based on the analysis results for 6-mm thickness.

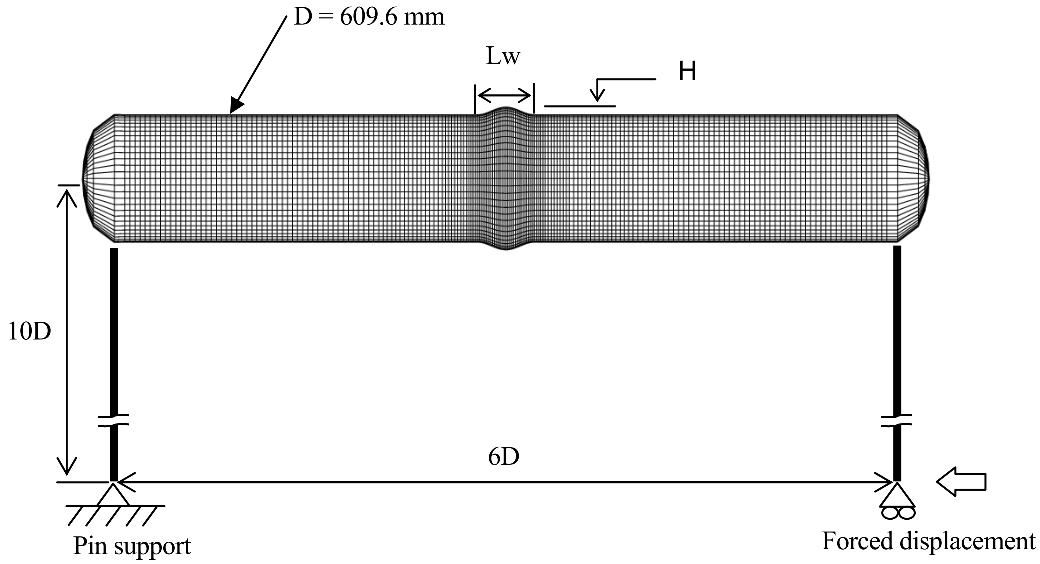


Fig. 5 FEM model.

Table 3 Numerical parameters

Thickness	Wave Width	Wave Heigth
t (mm)	W (mm)	H (mm)
6	150(1.0Lw)	6(1.0t) – 24(4.0t)
	225(1.5Lw)	6(1.0t) – 30(5.0t)
	300(2.0Lw)	6(1.0t) – 24(4.0t)
9	270(1.5Lw)	18(2.0t) – 45(5.0t)

2.5 Ultimate bending angle of the pipe module

As mentioned above, this pipe module must not suffer crack initiation or water leakage when a large fault displacement occurs. As the bending deformation increases, the pipe module deforms as shown in Fig. 6. At first, a convex portion like cymbals is generated in the center of the pipe, which then grows and the inner walls of both sides of this portion finally make contact. When this portion starts to decline, a very complex combination of compressive and tensile deformation occurs in the pipe module and the probability of crack initiation increases rapidly. Therefore, we defined the ultimate bending angle θmax of this pipe unit as the angle when both sides of inner wall plates make contact.

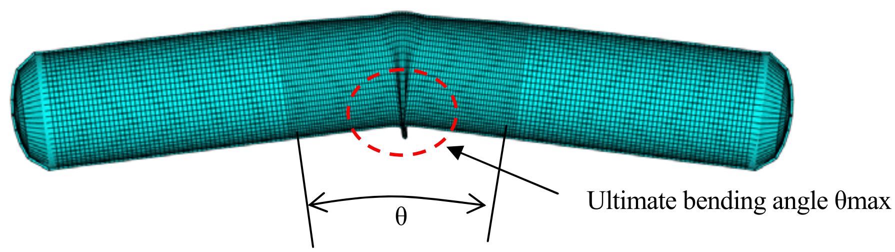


Fig. 6 Ultimate bending angle

2.6 Analytical results

Figure 7 shows the analysis results of the parametric study, indicated as the relation between the bending angle and bending moment of the pipe module.

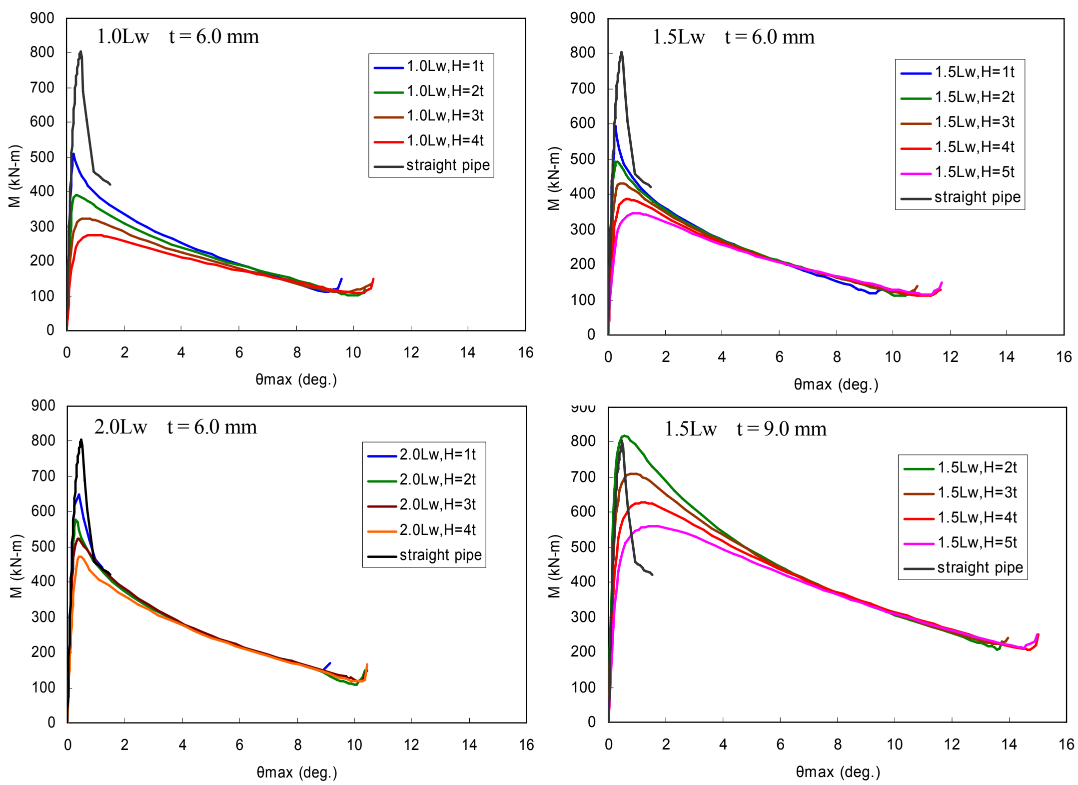


Fig. 7 Relation between bending angle and bending moment of pipe module

As shown in Fig. 7, the maximum bending moment of the seismic-protected pipe module is smaller than that of the straight pipe. This indicates that the seismic-protected pipe module deforms more easily than the straight pipe. This tendency is more remarkable for a pipe with high wave height. However, the trend of bending moment after the maximum point is not affected by the wave height and gradually becomes smaller up to the ultimate bending angle. This means that the pipe shape after buckling is controlled by the wave width, not the wave height.

According to the numerical analysis for a pipe of 6 mm wall thickness, the ultimate bending angle is not sensitive to the initial wave height of the pipe module as shown in Fig. 8. That is, the maximum value of the ultimate bending angle becomes almost constant even if the initial wave height of the pipe module is higher. On the other hand, the wave width of the pipe module affects the ultimate bending angle. The case of wave width $L = 1.5L_w$ gives the maximum value of the ultimate bending angle.

Next, a numerical analysis for a pipe of 9 mm wall thickness and $L = 1.5L_w$ wave width was conducted. As shown in Fig. 9, the ultimate bending angle of the pipe is about 30% greater than that of the pipe of 6 mm wall thickness. The wall thickness of 9 mm is the maximum for the seismic-protected pipe module for 600-mm diameter pipelines, since the bending strength of the pipe module will be greater than that of normal straight pipes if a wall thickness of over 9 mm is adopted for the seismic-protected pipe module.

Table 4 summarizes the numerical results. Based on the numerical study above, the optimal specifications for a 600-mm diameter seismic-protected pipe module were designed. The wall thickness is 9 mm, the wave width is 270 mm, the wave height is 36 mm (= 4t) and the ultimate bending angle is 14.7 degrees.

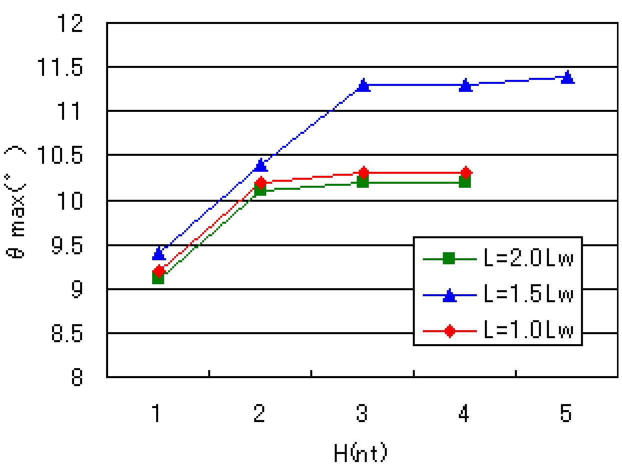


Fig. 8 Ultimate bending angle for 6-mm thickness pipe

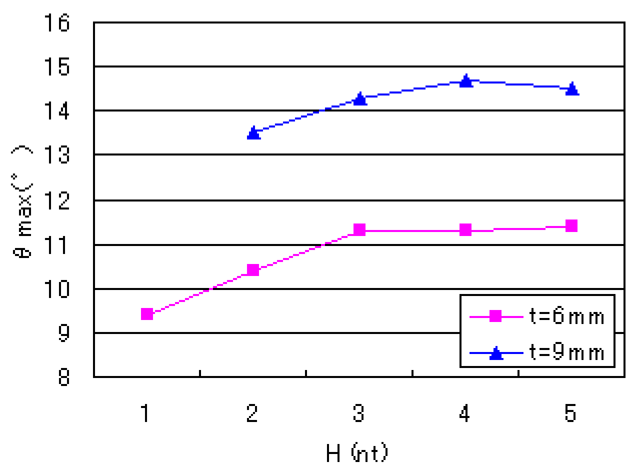


Fig. 9 Ultimate bending angle for 1.5Lw wave width pipe

Table 4. Summary of analysis results

Thickness T (mm)	Wave Width W (mm)	Wave Height H (mm)	Maximum bending angle θmax (deg.)
6	150 (1.0Lw)	6 (1.0t)	9.2
		12 (2.0t)	10.2
		18 (3.0t)	10.3
		24 (4.0t)	10.3
	225 (1.5Lw)	6 (1.0t)	9.4
		12 (2.0t)	10.4
		18 (3.0t)	10.6
		24 (4.0t)	11.3
		30 (5.0t)	11.4
	300 (2.0Lw)	6 (1.0t)	9.1
		12 (2.0t)	10.1
		18 (3.0t)	10.2
		24 (4.0t)	10.2
9	270 (1.5Lw)	18 (2.0t)	13.5
		27 (3.0t)	13.7
		36 (4.0t)	**14.7**
		45 (5.0t)	14.5

2.7 Seismic behavior of the pipeline system

An FEM analysis for the 600-mm diameter buried pipeline across an active fault with the seismic-protected pipe modules was conducted. The assumed fault conditions were:

(a) Fault type — Class A & Reverse Type
(b) Fault displacement — 3.5 m (Mean value for Class A faults)
(c) Angle of inclination of fault — 45°

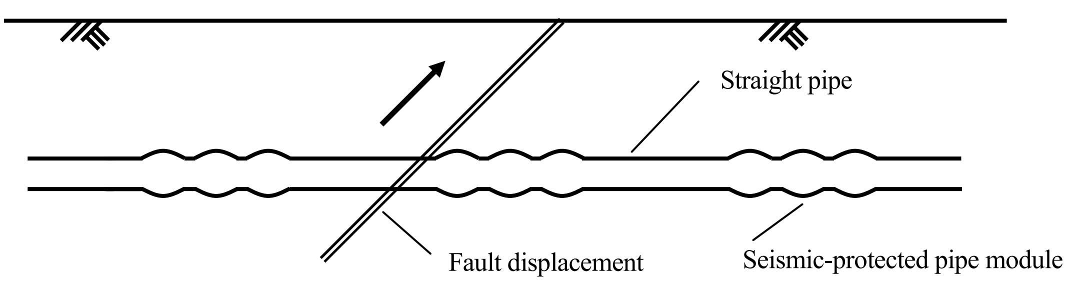


Fig. 10 Analytical model of buried pipeline

Figure 11 shows an example of numerical analysis for the buried pipeline system. When the fault displacement is generated in the 45° direction along the fault plane, the pipe modules nearest to both ends of the straight pipe deform first. As the deformations of nearest modules grow larger, the next pipe modules located on the outside of these modules begin to deform and the deformations spread to the outer pipe modules one after another. After the deformations spread to the last modules, buckling occurs in the nearest bent pipe modules and it also propagates to the outer pipe modules as the fault deformation grow larger. Since the seismic-protected pipe module can bend up to the angle of 14.7 degree, bending deformation of the pipeline up to the angle of 45 degree is acceptable when we settle a pair of 3 modules both sides of fault plane as shown in Fig.11.

In this way, a pipeline which consists of several seismic-protected pipe modules and straight pipes can absorb the large displacement of an active fault. Therefore, we can construct a seismic-protected pipeline across an active fault at lower cost by using these pipe modules.

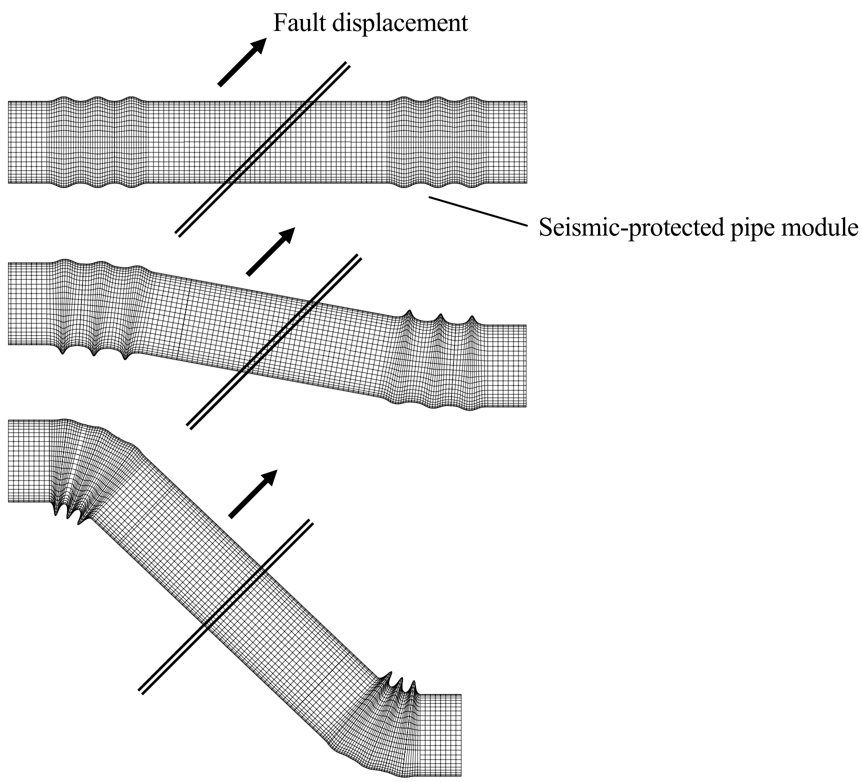


Fig. 11 Analytical behavior of seismic-protected pipeline

3. FUNTURE ISSUES

We will start an experimental study as outlined below to investigate the bending performance of the seismic-protected pipe modules in this year.

3.1 Experimental device

Structural experiments will be carried out with the loading device of 300 kN as shown in Fig. 12. It will bend the test specimen until a crack occurs in the pipe module or until the maximum stroke of the loading device (500 mm) is reached.

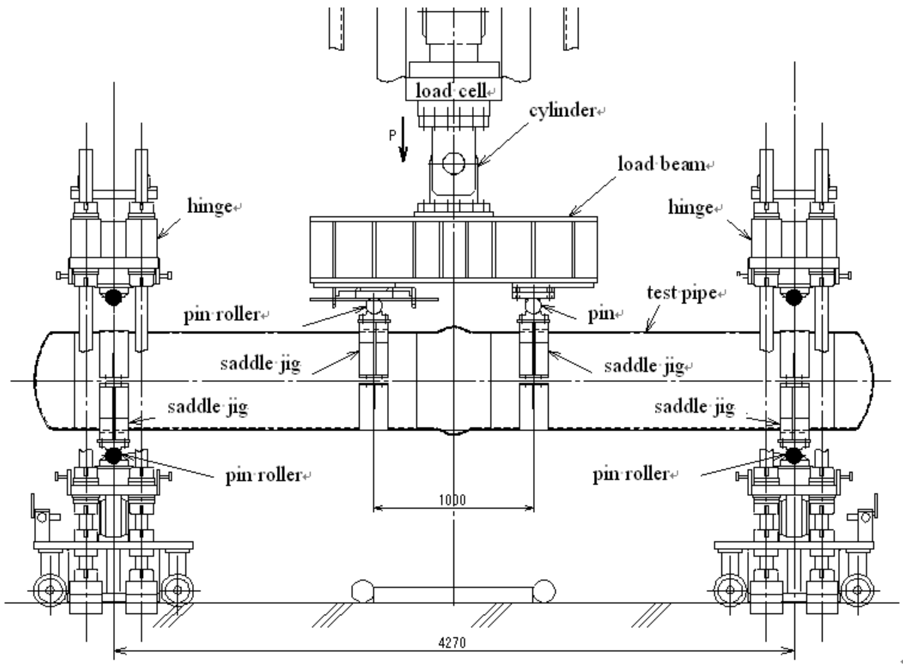


Fig. 12 Experimental device.

4. Conclusion

In this study, we developed a seismic-protected pipe module for pipelines across active faults. Based on a numerical study, the characteristics of deformation of these modules were clarified and the optimum specifications for the seismic-protected pipe module were decided. The required seismic performance of pipelines across active faults can be accomplished at lower cost by using these modules and straight pipes. In the next study, we will conduct experiments to investigate the bending performance of the modules.

REFERENCES

[1] JWWA, 2009, *Guideline of Seismic Design for Water Facilities.*

Seismic Upgrades of Portland Water Bureau's Water Supply System

Keith Walker[1]

ABSTRACT

The City of Portland Water Bureau (PWB) is currently undertaking a series of strategic hardening projects of its primary water supply system to increase redundancy and reduce vulnerability, therefore maintaining level of service throughout a seismic event.

Since 1895, the Bull Run Watershed serves as the primary source of water for the City of Portland and environs. The watershed serves approximately 800,000 people. It is located 26 miles east of Portland, within the Mt. Hood National Forest.

The water supply system from the Bull Run Watershed consists of three conduits, numbered 2, 3, and 4; built in 1911, 1921 and 1953, respectively; and several in town reservoirs totaling approximately 240MG.

Specific attention will be given to two water supply system projects:

- The Sandy River Conduit Relocation Project will allow two of three conduits (66" and 52" diameter) to pass under the Sandy river in a tunnel approximately 450 feet long, and 90 feet deep. Currently, the two conduits cross the river on a 100 year old pin truss bridge. The construction cost of the project is approximately $21 million.

- The Water Bureau's long-range water storage plan includes the construction of four 50-MG reservoirs and a smaller 20-MG on Powell Butte, purchased by the City in 1925 for this purpose. The first underground reservoir was built in 1979-1980 and became operational in 1981.

PWB is advancing its plans for a second underground 50-million gallon (MG) reservoir at Powell Butte Nature Park, by the year 2013.

A comparative analysis of the existing and proposed 50MG reservoirs will highlight the advances in structural seismic design over the past 30 years.

[1]Keith Walker, Senior Engineer, Manager, Storage and Pump Stations Program, Engineering Services Group, Portland Water Bureau, 1120 SW 5th Avenue, Suite 600, Portland, OR 97204,
Phone: (503) 823-7238, Fax: (503) 823-4500
Email: Keith.Walker@ci.portland.or.us

1. SANDY RIVER CROSSING

SYSTEM DESCRIPTION

The City of Portland is served by two water sources:

- The Bull Run Watershed, located approximately 25 miles east of Portland. Water is conveyed to Portland by three Conduits numbered 2, 3, and 4, built in 1911, 1921 and 1953, respectively (Conduit 1 was abandoned in 1952).
- The Groundwater system, located within the City of Portland on the south shore of the Columbia River, east of Interstate I-205.

The average system winter demand is approximately 100 million gallons per day (MGD). The peak summer demand exceeds 185 MGD, which is the maximum capacity of the existing conduit system.

Conduits 2, 3, and 4 transport water from the Bull Run Headworks to Powell Butte Reservoir and the Open Reservoirs at Mount Tabor (to be decommissioned in 2014). The nominal capacities of Conduits 2, 3, and 4 are 50, 75, and 100 MGD respectively.

The hydraulic flow in the conduits includes both open channel and full port pressure flow, depending on the location along the conduit and the operating condition.

The Groundwater system has a nominal capacity of 72 MGD. Therefore during the winter, the Water Bureau can meet demand with any two of the conduits out of service, and the Groundwater system in operation.

The Sandy River Crossings are two of a number of bridge and trestle crossings for the conduit system, where the conduits rise above grade or cross a water body on the journey to Portland. Other bridges include the Bowman Roadway and Pipeline Bridge (Conduits 2 and 4), , Larsens Roadway and Pipeline Bridge (Conduit 3), and the Headworks Pipeline Bridge (Conduit 3).

There are two Sandy River conduit bridges, one carries Conduits 2 and 4 (Figure 1), and the other carries Conduit 3. The Conduits #2 and #4 Pipeline Bridge, is an 1893 vintage through truss bridge just 20 feet upstream from the Lusted Road vehicle bridge. The bridge is 300 feet long and 16 feet wide. The bridge is estimated to weigh approximately 110 tons. Conduit #3 crosses the Sandy River on a separate bridge located about ½ mile downstream.

Figure 1, Sandy River Bridge 2, carrying Conduits 2 and 4; with adjacent Lusted Bridge (automobile/pedestrian)

Together, these Sandy River bridge crossings pose the greatest seismic risk for the entire Portland water supply system in terms of the potential impact for extended loss of water service to customers. The crossings are also subject to failures from flood scour of the bridge footings and from lahars originating on Mt. Hood, as well as being vulnerable to malevolent acts.

In November of 1995, a small landslide disabled conduits 2 and 4 on the Bull Run River bridge crossing. Portland had to activate its backup Groundwater System supply to maintain supply.

ANALYSIS HISTORY

A major goal of the analysis has been to assure that the water system includes one continuous "hardened" conduit run from the Bull Run reservoirs to the storage reservoirs and transmission system in town. As part of meeting this goal, the tunneling or burial of at least one conduit crossing under the Sandy River has been recommended. The project will reduce system vulnerability to a variety of factors including seismic hazards, flood hazards, and vandalism.

In 1996, the Sandy River Bridges, along with the other bridges and entire conduit system, were analyzed [1] for vulnerabilities should an earthquake, landslide, or human caused problem occur. Specifically, the bridges were reviewed in accordance with American Association of State Highway and Transportation Officials (AASHTO) provisions using the following loads:

- Seismic, 500 year event with PGA of 0.29g
- Wind, 50 year event, 80 mph wind speed
- Snow, 50 year event, ground snow load of 25 pounds per square foot.

The analysis concluded that the bridge superstructures are designed to outdated codes and are vulnerable to failure from earthquake induced ground shaking and, in some cases extreme wind pressures. Further, the Sandy River Bridges are vulnerable to damage in 200 year floods and volcano induced debris flow (500 year event).

Studies were conducted to propose a strategy in staging of the improvements for the entire conduit system [2]. Further studies specifically addressed the Sandy River Crossings:

- The System Vulnerability Assessment (SVA) (2000) rated the Sandy River bridge foundations as the highest risk in the system due to flood and scour.
- The Infrastructure Master Plan (2000) identified the Sandy River Crossing as a key vulnerability reduction.
- The Security Vulnerability Assessment (SecVA) (2003) rated the exposed conduits as one of the most critical security risks, with these crossings as the highest risk.

The remoteness of this Sandy River crossing, the potential damage to the conduits from natural or human sources, and the importance of the conduits in delivering Portland's water, contributed to the decision to address this area.

Budget

The Sandy River Conduit Relocation Project is funded for design and construction in the Water Bureau Capital Improvement Program in the years 2006-2011. The current estimated budget is $ 12.5 M, which includes a contingency for project unknowns. Preliminary engineering for this project was performed between 1996 and 2001 with follow-up in 2003. The project was placed on hold between 2003 and 2005, as funds were not available. Portland Water Bureau's Engineering Planning Section created a Basis of Design report [3] for this project in FY 2005-2006.

BDR PROJECT DESIGN RECOMMENDATIONS

The conduits on the pipelines bridges at the Sandy River are the only means by which Bull Run water reaches the City of Portland. There are risks to these conduits due to earthquakes, floods, lahars (debris flows triggered by volcanic activity), and man-made malevolent actions.

Certain practical concerns also supported tunneling Conduits 3 & 5 (future) first. The Bureau of Land Management and the State had established the "Sandy Wild and Scenic River and State Scenic Waterway Management Plan" in 1992. There was concern among Portland Water Bureau management that the "Wild and Scenic" designation and the implementation of the anadromous fish listing under the Endangered Species Act (ESA) would make it increasingly difficult to do a burial project, especially at Conduit 3 crossing which is inside the Wild and Scenic area (Conduits 2 & 4 crossing is just outside the Wild and Scenic area). As a result of this and other factors, the preliminary engineering stakeholder process had favored tunneling Conduits 3 and 5 first.

Alternately, some Portland Water Bureau management staff preferred the alternative of tunneling Conduits 2 & 4 first. This alternative provides higher capacity and reliability in the short run (Conduits 2 and 4 have a nominal capacity of 150 mgd vs. 75 mgd for Conduit 3). This alternative is estimated to be about $2 M less expensive, has fewer environmental impacts, and all the non-river bottom right of way is currently in possession of Portland Water Bureau.

Post September 11, 2001, all water systems were required to review their security vulnerabilities. The Security Vulnerability Assessment (2003) rated these exposed conduits as one of the most critical security risks, noting that the Sandy River Crossing “presents one of the most attractive targets due to its accessibility and visibility to the public.” Conduits 2 & 4 were identified as more vulnerable than Conduit 3 due to their accessibility, and all above ground conduit sections were considered highly vulnerable.

Finally, in the last few years, a number of the factors have shifted proposed Conduit 5 farther into the future. Annual Average water demand declined after the 2001 economic downturn and was below 100 MGD in 2004. By 2005, Portland Water Bureau was working closely with regulatory agencies on a Habitat Conservation Plan to mitigate fish impacts from the water system, and Portland Water Bureau had a much better understanding of likely water needs for complying with environmental concerns.

The key findings presented in BDR report are the quantified benefits of replacing the bridge that carries Bull Run Conduits 2 and 4 over the Sandy River with a tunnel. The benefits are expressed in reductions of system days of lost water service immediately following an earthquake that are attributable to replacing the bridge by a tunnel. The benefit of constructing the proposed tunnel under the Sandy River for Conduits 2 and 4 is:

- 29.3 days of average daily demand (@ 100 MGD) for earthquakes greater than 0.32g Peak Ground Acceleration.
- 6.3 days of average daily demand (@ 100 MGD) for earthquakes in the range 0.16g to 0.32g.
- No benefit for earthquakes below 0.16g. The estimated cost of constructing the tunnel is $12.5 million dollars.

The Sandy River Crossing Project Basis of Design Report (BDR) confirmed the following major design recommendations:

1. Security: Install buried pipes and provide standard security protection at access vaults.
2. Pipe Sizing: Install two 72” pipes to assure adequate transmission capacity for future needs.
3. Construction Method: Excavate tunnel using conventional means. If another method is proposed it cannot be more expensive, extend outside Portland Water Bureau property, be more difficult to permit, or use more than two pipes.
4. Geotechnical: Undertake additional geotechnical exploratory work to assure contractors do not see the project as too risky, and to assure stability of the west stream bank.
5. Interconnections/Valves on main conduits: Interconnections and isolation valves have been provided at Larson's and Hudson’s interties so no new interconnections are recommended.
6. Historic Bridge Disposition: Regulators see the historic pipeline bridge as integrally related to the project. Portland Water Bureau would prefer to move the bridge to reduce liability and will work with SHPO to arrange for an appropriate option for bridge disposition.
7. Permitting Strategy: Because there are at least 16 permitting agencies involved, for grant purposes and smooth project scheduling an Environmental Assessment is recommended. USACE 404/DSL removal permits will be required, and a biological assessment is likely.
8. Backfill in Tunnel: Pipes will be installed in a horseshoe shaped tunnel with backfilled concrete for earthquake resistant stability at lower cost.
9. Tunnel Access: Tunnel access will be provided through at grade vaults and then through the tunnel piping itself. The contract will include the tunnel inspection for the first 10 years.

10. Alignment: The East shaft location is in the 100-year flood plain. Design will be a hardened shaft access. Instability of the west riverbank has resulted in a slightly longer tunnel on the West side.
11. Procurement Method: An alternative contracting method, Design-build (DB) or Construction Manager/General Contractor (CM/GC), was recommended to ensure a high quality contractor, fast track scheduling, and improved communication between design and construction firms. Portland Water Bureau selected the Design Build process.

In October, 2005, the Portland Water Bureau Administrator was briefed on these project concerns. Following the briefing, the Administrator, with concurrence of the Chief Engineer, determined that the Sandy River Crossings project would go forward and that Conduits #2 and 4 would be buried (tunneled) first. It remains the assumption that Conduit 3 will be tunneled at some time in the future.

GEOTECHNICAL WORK

Geotechnical work was performed as part of the Feasibility Study and preliminary engineering Report phases. To minimize risks associated with tunneling under the river, the depth was determined to provide approximately 40 feet of Sandy River Mudstone below deposited material subject to scour, at the lowest point of the river, while maintaining the tunnel invert at an elevation15-20 feet above the harder rock of the Rhododendron Formation below.

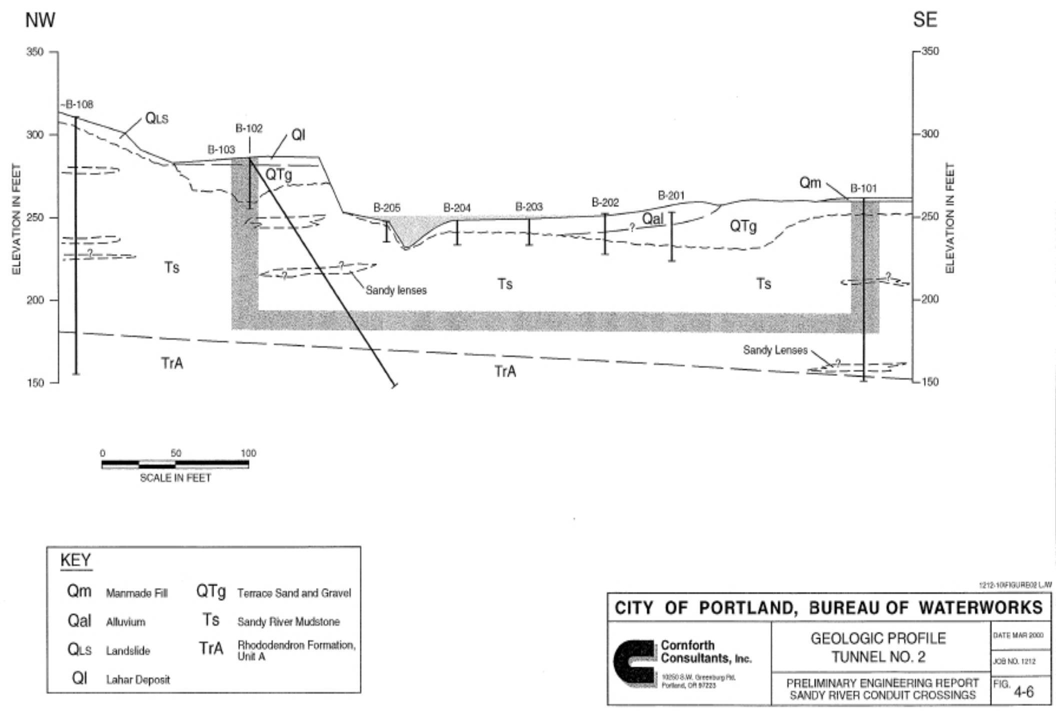


Figure 2, Geologic and Tunnel Profile

DESIGN BUILD

Portland Water Bureau's selection in using a DB method for the project includes reduction in project risk, duration, constructability and an allowance for innovation, high quality contractor selection for this complex project, and establishment of cost. For a tunnel, Portland Water Bureau prefers to have close coordination of design and construction to ensure constructability since tunnel work is a specialty area with high risks. Portland Water Bureau also has an interest in executing the project in a shorter time frame for capital budget financial reasons.

Construction crews started work in the Sandy River canyon on the first phase of the project in late 2007. Kiewit Pacific Co., under the Portland Water Bureau's first-ever design-build contract, started construction a 434-foot long tunnel under the Sandy River and relocate water conduits No.2 and No. 4. The work site is located within the bureau's Sandy River Station facility. The work consists of the following:

- Excavation of the large east side tunnel, approximately 30 feet in diameter, to allow the tunnel boring equipment down into the tunnel.
- Vertical drilling two small west side tunnel risers
- Rock boring and shotcrete for the under river portion.

The construction method recommended by the PE Report is called conventional tunneling. In this method, vertical shafts are created with shoring/reinforcing. These are fairly large in diameter to permit crews and equipment to reach the bottom of the shafts. (proposed 87 feet depth at entry end, 101 feet at west end). Tunnels are then dug out a few feet at a time (typically 4 to 8 feet) and steel rib shoring is placed in the tunnel with a reinforcing lattice, and then shotcrete is overlayed. The tunnel was dug with a road header machine (portable boring tool) with supplemental manual work. This method was used to create an arched tunnel to house two conduits in the same tunnel and is appropriate to the soils anticipated for the Conduit 2 & 4 crossing.

The preferred design for Tunnel No. 2 consists of twin side-by-side, 72-inch diameter steel pipes, each with a hydraulic capacity of 165 mgd to provide additional flow beyond the current capacity of existing Conduit 4 (currently 100 mgd). The tunnel mining horizon depth criteria is approximately 40 feet below the lowest point of the river channel. Ground conditions are bedrock that is soft and degradable. A horseshoe shaped tunnel approximately 10 feet high by 16 feet wide, which is to be backfilled with cellular grout.

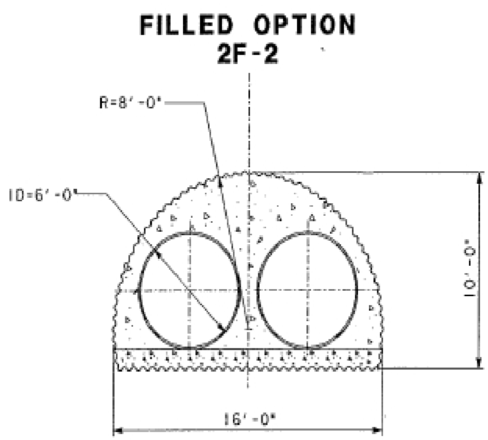


Figure 3, Tunnel Cross Section

Tunnel No. 2 (for Conduits 2 and 4) will cross diagonally under the path of the existing Conduit 2 and 4 bridge. The shaft on the west side of the river located on Water Bureau property will be the secondary shaft. The west shaft will be approximately 20 feet in diameter and 105 feet deep. The tunnel will be a minimum of 40 feet below the lowest riverbed material. Most construction activity will occur at the shaft on the east side of the river, which for this tunnel will be located in the Water Portland Water Bureau's Sandy River Station maintenance yard. The east shaft is approximately 30 feet in diameter and 80 feet deep.

SANDY RIVER CROSSING CONSTRUCTION STATUS

The project's official groundbreaking took place October 3, 2008. Initially, Kiewit Pacific Co. has prepared the site for construction and has excavated the east shaft. Round corrugated steel bulkheads were used to allow excavation through the sandy topsoils.

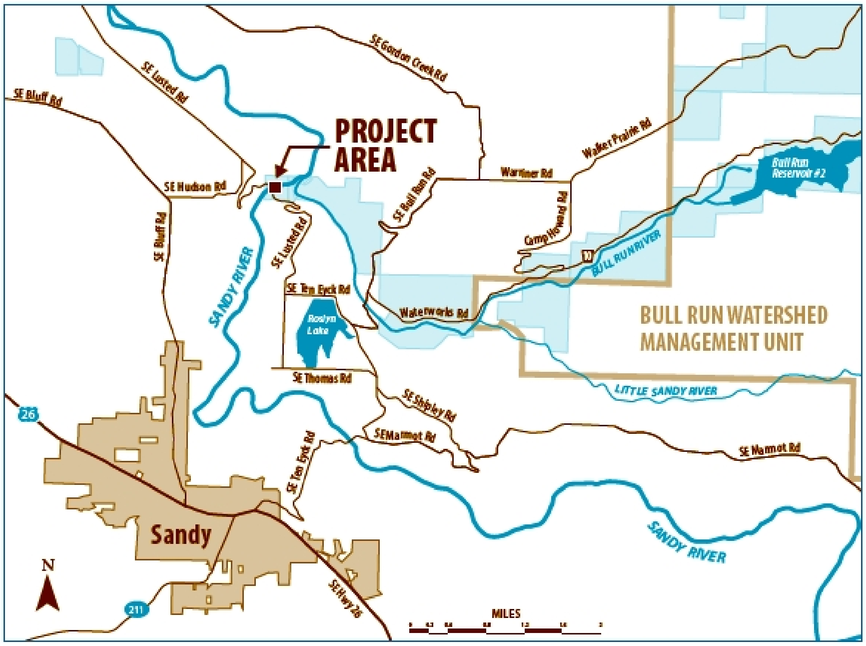


Figure 4, Project Location

As of September 2009, the Kiewit Pacific Co. and its subcontractors have completed the horizontal tunnel drilling portion of the crossing, as well as excavation of the shaft on the east side of the river, and two riser shafts on the west side. The conduits remain to be placed in the tunnel. Upon completion of the conduit placement, temporary bulkheads will allow hydrostatic testing of the crossing piping, upon successful completion of testing the tunnel will be completely backfilled with cellular grout.

When completed, the pipeline bridge, which is located next to the auto bridge will be removed from service. Construction is estimated to last for approximately 2 more years.

2. POWELL BUTTE 2 RESERVOIR

FACILITY DESCRIPTION AND ANALYSIS HISTORY

Powell Butte, is located in Southeast Portland. The butte (an isolated hill) is an extinct cinder cone volcano, rising near the headwaters of Johnson Creek, an urban creek. The area encompasses 608 acres of meadowland and forest.

In 1925 the City of Portland purchased the land for future water reservoirs. In the mid-1970s the Water Bureau prepared a development plan for Powell Butte that called for the construction of four 50-million gallon (MG) underground reservoirs to be located at the north end of the butte. In 1981 the first, 50 MG reservoir was placed in operation and serves as the primary distribution hub of the Water Bureau's distribution system.

A Masterplan outlining the future long term storage strategy was developed for Powell Butte in 1985. The study was subsequently updated in 1995-6 in conjunction with Portland Parks and Recreation (PP&R), and further updated in 2003 with the Conditional Use Master Plan (CUMP), which specifically addressed the design of the second 50 MG reservoir as well as water system and park improvements. A seismic study [4] was also completed in 2007 which also confirmed the Water Bureau's storage strategy.

LONG TERM 2 ENHANCED SURFACE WATER TREATMENT RULE

The Long Term 2 Enhanced Surface Water Treatment Rule (LT2) was introduced by the U.S. Environmental Protection Agency (EPA) in 2000, and adopted in 2006, as part of the implementation of the Clean Water Act of 1996. The purpose of the LT2 rule is to reduce illness linked with the contaminant Cryptosporidium and other disease-causing microorganisms in drinking water.

With regard to Portland's storage, LT2 requires that all public water systems that store treated water in an open reservoir, such as those at Mt. Tabor and Washington Park, do one of the following:

- Cover the finished water storage facility
- Treat the water again before it goes to customers
- Stop using the storage facility to store finished drinking water

The City of Portland's system currently has five uncovered facilities:

- East Side:
    - Mount Tabor Reservoir #1: 12 MG (million gallon)
    - Mount Tabor Reservoir #5: 49 MG
    - Mount Tabor Reservoir #6: 75 MG
- West Side:
    - Washington Park Reservoir #3: 16.4 MG
    - Washington Park Reservoir #4: 17.6 MG

On March 25th, 2009, the Portland Water Bureau notified the EPA [5] of the intent to replace the East Side Storage at Mt. Tabor with the 50 MG of additional storage at Powell Butte, along with an additional 15 MG of storage at Kelly Butte. West Side storage will be replaced with 15 MG covered storage at Washington Park. Each improvement element will require transmission and other piping improvements. Detailed information is included in the report entitled, 'LT2 Storage Recommendation.' [6]

The State of Oregon and the EPA subsequently approved the schedule for meeting the open reservoir requirements by April 1, 2009. In order to decommission the open reservoirs, it is necessary to build covered storage consisting of buried reservoirs, and then disconnect the open reservoirs from the public water system.

The plan for the second 50 MG reservoir at Powell Butte (PB2) calls for two phases. Phase 1 entails site preparation and preliminary excavation of the reservoir site.

The first phase has started in August 2009, and will take about 6 to 8 months to complete. An estimated 100 truck trips per day for a total of approximately 30,000 trips to and from Powell Butte during Phase 1.

Phase 2 will start in summer 2010 will be final excavation, and design and construction of the buried concrete reservoir, pipes, vaults, emergency overflow, and a number of park improvements.

The design is to be a 50 MG rectangular reinforced concrete reservoir with two 25 MG cells. Emergency overflow is estimated to by a maximum of 60 MG per event (225 MG per day). The emergency overflow shall be channeled to Johnson Creek to the south of the Powell Butte.

GEOLOGIC AND SUBSURFACE CONDITIONS

Earthquakes in the Pacific Northwest occur largely as a result of the collision between the Juan de Fuca and the North American plates, and associated volcanic activity. These two tectonic plates meet along a mega thrust fault called the Cascadia Subduction Zone (CSZ). The CSZ runs approximately parallel to the coastline from northernmost California to southern British Columbia. The compressional forces that exist between these two colliding plates cause the denser oceanic plate to descend, or subduct, beneath the continental plate. This process leads to contortion and faulting of both crustal plates.

Previous studies and other documents by the Portland Water Bureau have investigated whether the Powell Butte Reservoir 1 is in need of seismic upgrade, or has been constructed to the correct standards to withstand earthquake activity with acceptable risk to the structure of the reservoir.

There have been some significant earthquakes in or near the Portland area in the last 150 years. From the USGS records, two in particular stand out due to their proximity to Powell Butte and their relative magnitude: On October 12, 1877 a quake occurred with its epicenter only a few hundred feet north of Powell Butte. (A quake's epicenter is the point on the Earth's surface that is directly above the focus, or hypocenter, of the earthquake.) It has been classified as a *shallow* earthquake, though this could still mean several kilometers deep. The second of interest occurred on November 5, 1962, only several thousand feet from Powell Butte, and was at about 18 kilometers (about 11 miles) deep. Both were estimated at 5.3 on the Richter scale. A 5.3-magnitude quake is considered to be of moderate magnitude.

It is important to note that the 1877 quake was not measured with scientific devices. The science of seismology had not matured to its current state at that point. Data concerning this earthquake came from news accounts and first-hand testimony of people who experienced the quake, and values for location and magnitude were estimated later. As a result, there is significant uncertainty in the value for the magnitude of the 1877 earthquake.

The construction specifications for the existing Reservoir include a Geotechnical Investigation by Shannon and Wilson, Inc., which discusses the geotechnical background of the Reservoir area [7]. Shannon and Wilson's investigation discussed the seismicity of the Portland area.

Shannon and Wilson proposed that the relatively short period of record for earthquakes in the Portland Metro area, and close environs, makes it unlikely that the largest probable earthquake has yet been recorded. Their analysis shows that the largest quake that could reasonably be expected to occur is a magnitude 6.0 event. For design purposes, they proposed a 6.5 magnitude quake to be conservative in the design of the structure.

The Oregon Department of Geology and Mineral Industries (DOGAMI) in coordination with METRO (The Portland Metropolitan area regional government), developed an earthquake hazards map of the State in 2000. The primary factors considered in the study included ground motion amplification, liquefaction susceptibility and slope instability. The City's GIS system incorporated this information into the system. The map, Figure 5, shows the relative earthquake hazard for the Powell Butte Area.

While areas nearer to downtown Portland, and particularly in the West Hills, may be at more risk from an earthquake, the information from the DOGAMI and METRO study indicates that Powell Butt is at low risk of damage from and earthquake.

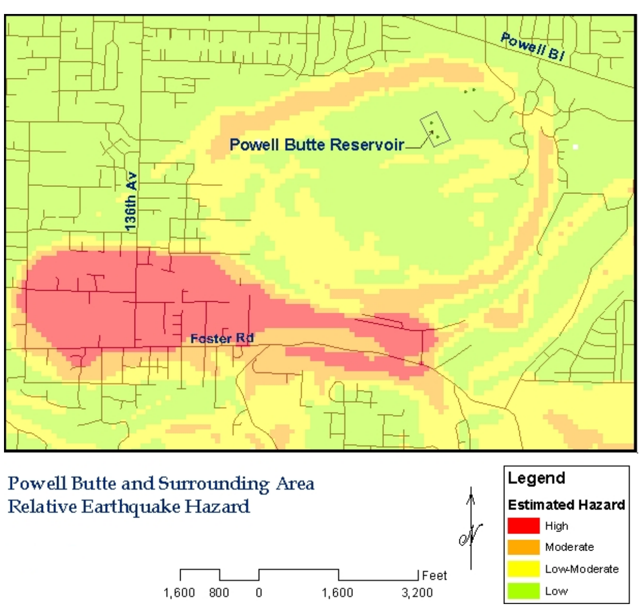


Figure 5, DOGAMI Relative Earthquake Hazard

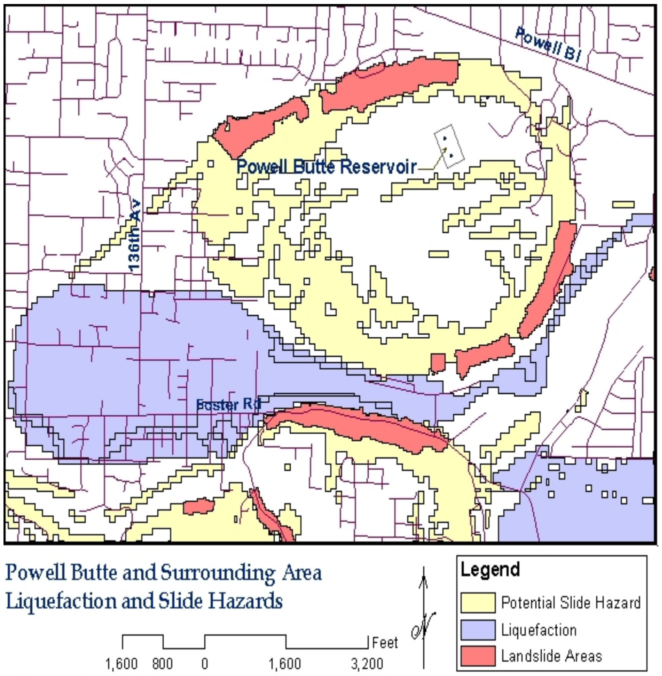


Figure 6, DOGAMI Liquefaction and Land Slide Hazards

The next map, Figure 6, shows the Powell Butte area and highlights the areas of risk due to liquefaction and probable land slide hazard. This information is also from the DOGAMI research. While landslides can occur at any time, particularly with heavy rain, liquefaction is typically a result of seismic activity.

The maps shows that the area near the Reservoir is relatively free from liquefaction and slide hazards. The proposed Powell Butte 2 reservoir, though not indicated on the maps, is west and adjacent of the Powell Butte 1 reservoir.

In preparation for the construction, a preliminary geotechnical report was prepared [8]. The evaluations characterized site soils and groundwater through explorations consisting of rotary auger borings with Standard Penetration Testing (SPT) borings, Cone Penetrometer (CPT) soundings, and laboratory tests run on collected soil samples.

Task One of the geotechnical report was comprised mainly of field work including site reconnaissance and subsurface explorations.

Soils samples collected during the subsurface exploration were used to interpret and group the subsurface soils into units for discussion and engineering evaluation purposes. From the ground surface downward, the units are as follows:

- Fill: loose to medium dense silt and sand, trace fine gravel; when present, ranged from one to four feet thick
- Loess: wind-blown loose to medium dense silt and fine sand; when present, ranged from one to nine and a half feet thick and had an average Standard Penetration Test N (SPT) value of 21 blows per foot (bpf)
- Residual Soil: loose to medium dense silt, sand and fine gravel; when present, ranged from one to 10.5 feet thick and had an average SPT value of 19 bpf
- Springwater Formation: very dense silty fine to coarse gravel with sand and cobbles; extends in depth past all exploratory borings and likely continues to large relative depths, average SPT value of 71 bpf, but ranges widely between 23 to refusal (greater than 50 blows for 6 inches).

Based on the material types, measured relative densities, measured shear wave velocities, and lack of groundwater, liquefaction is not considered a potential hazard at this site. Additionally, liquefaction related subsidence or deformations, including lateral spreading is not expected.

For construction temporary cut slopes, a recommendation was that the excavation should be sloped at no steeper than 1.5 horizontal:1 vertical (1.5H:1V), provided perched groundwater and loose soil are not encountered.

In order to characterize the recurrence, magnitude, and source to site distance, the Consultant reviewed the information available from the United States Geological Survey (USGS) Probabilistic Seismic Hazard Analysis (PSHA) interactive deaggregation web site. The USGS deaggregation results provide earthquake magnitudes and distances that are considered to be the most significant contributors to the ground motion hazard for a particular return period and spectral acceleration period at the specific site. The dominant earthquake magnitudes and distances that contribute to the seismic hazard may vary with the ground motion return period and oscillator period. As specified by the current IBC code, ground motions were evaluated for the Maximum Considered Earthquake (MCE), which is equivalent to the 2,475-year return period ground motions, or motions with 2 percent probability of exceedance in a 50 year period.

Table I, Probabilistic Deaggregation

Summary of USGS Probabilistic Deaggregation for Ground Motions Having a 2475 Year Return Period

USGS Deaggregation	Bedrock PGA (g)	Principle Seismogenic Source	Contribution to Seismic Hazard (%)	Source to Site Distance (km)	Moment Magnitude (M_w)
2002	0.415	Shallow Crustal	88	5.6	6.2
		Deep Subcrustal	< 1	--	--
		CSZ	7	110	8.7
2008	0.441	Shallow Crustal	65	2.7	6.1
		Deep Subcrustal	13	65.0	6.9
		CSZ	16	111.8	9.0

The Table above shows two main components, the modal seismic event and the principle seismogenic sources, as determined within the USGS PSHA. The data regarding the modal event includes source to site distance, magnitude and mean PGA. Information regarding the principle seismogenic sources includes their relative contribution to the total ground motion hazard, magnitude, and source to site distance. Upon inspection of the data, it can be noted that the shallow crustal earthquakes generally dominate the seismic hazard for the PGA, (particularly for the 2002 PSHA). As a side note, the Grant Butte Fault contributes a full one-third of the total hazard under both PSHA's.

The Grant Butte Fault distance to the the project site is less than one-half km due north. The fault has two mapped traces that wrap around the butte: one across the north flank and one across the south flank. The fault is approximately 10 km long and is a northeast-striking high angle normal fault. The slip-rate has been estimated at less than 0.2 mm per year with the latest deformation within the last 750,000 years.

Specific tasks for second phase design geotechnical work are not finished and include:

- Seismic hazards
- Site specific seismic evaluation (IBC 2006 & OSSC 2007)
- Seismic coefficients and design response spectra
- Foundation recommendations (allowable bearing pressure, subgrade modulus for mats, differential settlement, etc.)
- Lateral earth pressure (static dead and live loads, seismic)
- Resistance to lateral loads (passive earth pressure, friction coefficient)
- Recommendations for subsurface drainage
- Recommendations for grading/backfill requirements

COMPARITIVE CONSTRUCTION OF POWELL BUTTE RESERVOIRS

The existing Powell Butte 1 reservoir (PB1) was analyzed in 1996 [9], in which the study addressed the criteria of the Uniform Building Code (UBC, 1997) which would require stricter design standards than the original design included, depending on how close a structure is to an active fault.

This information was further reviewed in 2002 [10] on then current seismic and structural standards. It was found to have several deficiencies that could compromise the storage in a seismic event. A risk analysis was further conducted by the Water Bureau in 2007.

Deficiencies include multiple failure mechanisms that are equally as likely to result in loss of functionality through flexural cracking during a seismic event. The following load cases were analysed for deficiencies, with their results listed:

- Full Tank, backfilled structure (buried), walls were adequately designed
- Full Tank, backfilled, under seismic event, minor repairable damage
- Empty tank, backfilled, borderline situation
- Empty tank, backfilled, under seismic load, repairable damage uner 500 year event, severe damage under 2,500 or 5,000 year design earthquakes.
- Full tank, no backfill: This condition is relevant during construction of PB2. The new design must be sited far enough away from PB1 so that it may remain in operation during construction. The excavation must also not undermine the PB1 foundation.
- Full tank, no backfill under seismic event. Can result in major damage.

The following are summaries of the recommendations from the Montgomery-Watson design analysis memorandum to upgrade the existing PB1 reservoir. The report is also an indicator as to the major structural differences and how they would be incorporated into Powell Butte 2 Reservoir. The Montgomery-Watson memorandum indicates that the seismic risk presented by these apparent deficiencies is low in each case.

- *Roof diaphragm chord steel is inadequate for current criteria and code. The long exterior walls have an indicated steel area of 19.74 square inches, where current criteria call for 41 square inches. The short exterior walls have 27.94 square inches, where current criteria call for 60 square inches.*
- *Concrete ductility is inadequate in the Reservoir. The design of the Reservoir called for half of the horizontal wall reinforcing to be stopped at all vertical construction joints, in order to better control shrinkage cracking and limit leakage. Additional shear reinforcing is normally applied to existing concrete structures by anchoring a shotcrete encased curtain of reinforcing to the wall. However, because the height to length ratios of the reservoir walls are very low, the risk due to lack of continuity of half of the wall horizontal reinforcing is judged to be low.*
- *Similarly, code requires that reinforcement in shear walls and diaphragms that are designed for a stress exceeding a nominal value be provided in two curtains or mats. The top and bottom reinforcing in the reservoir roof slab was designed for bending due to slab dead and live loads, and is not continuous as required by code. To bring the reservoir in compliance with this current ductility requirement would require addition of minor amounts of reinforcing over a considerable portion of the roof slab. The additional seismic risk due to this ductility deficiency is judged to be low.*
- *Building code requirements require two continuous reinforcing bars in the bottom of column strips of the flat slab construction that forms the reservoir roof. The ACI 318 Building Code and Commentary states that these bars are "integrity steel" which would give the slab some*

residual capacity in the event of a punching shear failure. These bars were not required by the building code and were not provided in the design. The risk of a punching shear failure under static or dynamic loads is very low.

The cost to upgrade the existing reservoir to current UBC standards would be $18.5 million. The Water Bureau concluded not to to upgrade the reservoir, in light of the increased redundancy of the new reservoir to be constructed, and the low risk of failure.

The recommendation was to commit finances to new structures and provide redundancy to new reservoir, i.e., PB2. The new facilities will allow more flexibility in case emergency repairs to the existing PB1 are required.

POWELL BUTTE 2 DESIGN AND CONSTRUCTION STATUS

Groundbreaking of the excavation portion of Powell Butte Reservoir 2, was on August 18th 2009. The excavation is expected to displace 350,000 cubic yards of material, and to last of six to eight month duration.

The second three-year phase of reservoir construction will begin summer 2010,after the rain season. A Request for Proposal for the design of the reservoir is pending.

Phase 2 of the construction will see the contractors will finish excavating the hole, build the 50-million-gallon rectangular concrete reservoir, remodel a restroom and interpretive center, add a new maintenance building and storage area (40,000 square feet), replace the caretaker’s house, improve the parking lot and maintain the park’s trails.

Also included with the work is an extension of Conduit 5, approximately 5000 linear feet of 84" to 90" inch diameter pipeline.

Estimated cost of the PB2 reservoir is estimated at $137.8 million, in 2008 dollars. Construction is expected to begin in 2010 and last through the end of 2013.The project is scheduled for completion in 2013.

REFERENCES

[1] KPFF Consulting Engineers, “Bull Run Bridge Maintenance and Improvements, Structural Analysis and Inspection Report with Recommended Improvements”, July 1996;
Harza, Squier Associates, Crane and Merseth, “Hydraulic, Structural, and Seismic Analysis of the Portland Water Supply Conduits”, 1996

[2] Portland Water Bureau, “Conduit Trestle System Vulnerability Reduction and Improvements”, August, 2000, A. Mohammadi & B. Nelson

KCM Engineers, “Conduit Isolation and Improvements Project Phase I – Pre Design”, March 1998 (Interties)

Black & Veatch, “Basis of Design Memorandum, Conduit Trestle System Vulnerability Reduction Project”, March 2005

[3] Portland Water Bureau, “Basis of Design Report, Sandy River Conduit Relocation”, 2006

[4] Portland Water Bureau, Project Concept Report for the Powell Butte Seismic Study, Joseph C. Howe

[5] Portland Water Bureau, Memo to EPA, http://www.portlandonline.com/water/index.cfm?c=49632&a=237777

[6] Portland Water Bureau, “LT2 Storage Recommendation”, March 25, 2009

[7] Shannon & Wilson, Inc., Geotechnical Consultants, Geotechnical Investigation – Powell Butte Reservoir – City of Portland, Oregon – Bureau of Waterworks, Revised September 11, 1979.

[8] Shannon & Wilson, Inc., Geotechnical Consultants, Geotechnical Design Report – Powell Butte Reservoir No. 2– Portland, Oregon – Portland Water Bureau, April 03, 2009.

[9] Montgomery Watson, (Mark Fordham, Denver Office), PWB Powell Butte Reservoir, January 2, 1996

[10] Ward, Eric, 'Powell Butte Reservoir 2 Preliminary Design Report', CH2M Hill, June 2002

Replacement of Ductile Iron Pipes Installed in Early Days

Kazuhiko Fujimura and Naoki Hosoi

ABSTRACT

TOKYO WATERWORKS SERVICE CO., LTD. (TSS) has conducted fundamental waterworks projects covering various operations and investigations in cooperation with the Bureau of Waterworks of Tokyo Metropolitan Government (the Bureau), and in the maintenance and control of water distribution facilities, TSS has been performing its part effectively, as a partner of the Bureau.

While the Bureau was formerly installing cast-iron pipes for water distribution network, it began to switch the applicable kind of pipes to ductile iron pipes gradually from around 1960 for the improvement of strength as they appeared and were available. The circumstances are as follows:

1) The times for switch are different according to pipe diameters.
2) Mortar lining for corrosion-proof has not been executed inside the ductile iron pipes adopted for several years since the beginning.
3) Ductile iron pipes have been adopted only for straight parts of water distribution network from the beginning, meanwhile for special fittings such as bend pipes, it was since FY 1973 that ductile iron pipes have been adopted.
4) Later, It was latter 1970's that polyethylene sleeve began to be installed to cover ductile iron pipes for preventing corrosion outside them.

As ductile iron pipes went on being replaced, they account for almost entire network which has extended approximately 25,700 km today. But the network which ductile iron pipes have been installed by early 1970's contains some types of incomplete pipes as mentioned above and it has such many problems on maintenance as water leakages or rusty water caused by corrosion inside/outside pipes, and risk of earthquake damages due to weakened pipes with decrease of pipe thickness and lack of restraint fittings.

Therefore the distribution pipe routes which have been installed by FY 1972 are called "early ductile iron pipe routes" in particular and distinguished from others because conventional cast-iron pipes have been mixed for special fittings on them.

To resolve these problems derived from early ductile iron pipes, for three years since FY 2001 TSS has analyzed the data based on the field surveys of approximately 2,500 points on early ductile iron pipe routes to prepare a basic database on deciding priority of replacing pipes in the distribution pipe routes.

The Bureau made a start on replacement project of early ductile iron pipes to improve earthquake-resistant water distribution network, and it is remarkable to set to work with scientific methods, so that this paper describes the contents and circumstances of the field surveys which TSS has conducted.

Kazuhiko Fujimura, Director, Planning and Coordination Division of Tokyo Waterworks Service Co., Ltd., Green Tower Building 6-14-1 Nishi-Shinjuku, Shinjuku-ku Tokyo, Japan 160-0023

Naoki Hosoi, Sectional Manager, Planning and Coordination Division of Tokyo Waterworks Service Co., Ltd., Green Tower Building 6-14-1 Nishi-Shinjuku, Shinjuku-ku Tokyo, Japan 160-0023

INTRODUCTION

TSS has been conducting waterworks activities as a partner of the Bureau.

TSS has been undertaking various maintenance and control services for water facilities which range from watershed forests to water supply equipments, providing a high level of waterworks technology and services with much technical knowledge and expertise than ordinary private companies. Especially, investigations and diagnoses of water distribution pipelines have been conducted for over consecutive 20 years, and it has been the foundation of TSS's scope of the work since its establishment.

In earnest, the Bureau started the replacement of ductile iron pipes installed in early days in FY 2005 that followed the replacement of cast-iron pipes (aged pipes) aiming at the improvement of earthquake-resistance and the prevention of water leakage and rusty water.

In advance of this replacement project, TSS has analyzed the data based on the field surveys of approximately 2,500 points on early ductile iron pipelines which have been conducted for three years since FY 2001 as part of investigations and diagnoses of water distribution pipelines to prepare a basic database on deciding priority of replacing pipes in the distribution pipelines.

This paper first makes a report of the TSS's role in Tokyo waterworks, next the result of the investigations and the analyses about early ductile iron pipelines conducted by TSS, and the process of deciding implementation of replacing pipes by the Bureau.

THE INTEGRATED OPERATION SYSTEM OF THE BUREAU

1. INTEGRATED OPERATION SYSTEM

TSS is placed as an administrative organization by Tokyo Metropolitan Government (TMG). It is defined as the organization on which TMG invests and supports human resources under the guidance and supervision.

In 2006 the Bureau launched "an Integrated Operation System" that the Bureau and TSS conduct fundamental operations of waterworks to provide safe and good water stably to Tokyo residents to the future with due responsibility, coping with public-minded and efficient management (Figure 1).

2. PARTS ASSIGNED TO THE BUREAU AND TSS

Under the integrated operation system the Bureau conducts fundamental operations such as development of the management policy and the facilities improvement plan.

TSS conducts important operations for the Bureau's waterworks management such as supervision and guidance of routine operations subcontracted by private companies and management of facilities. According to this policy it has been conducting various services such as maintenance and investigations for watershed forests, purification plants, water supply stations, water transmission and distribution pipes, inventory control of materials for water facilities against emergency and disasters and the hosting of trainings which aim at bringing up waterworks engineers.

In particular the results of "Investigation and Diagnoses of Water Distribution Pipelines" have become valuable information for appropriate maintenance of water distribution pipes, and for systematic and efficient renewal of distribution pipelines.

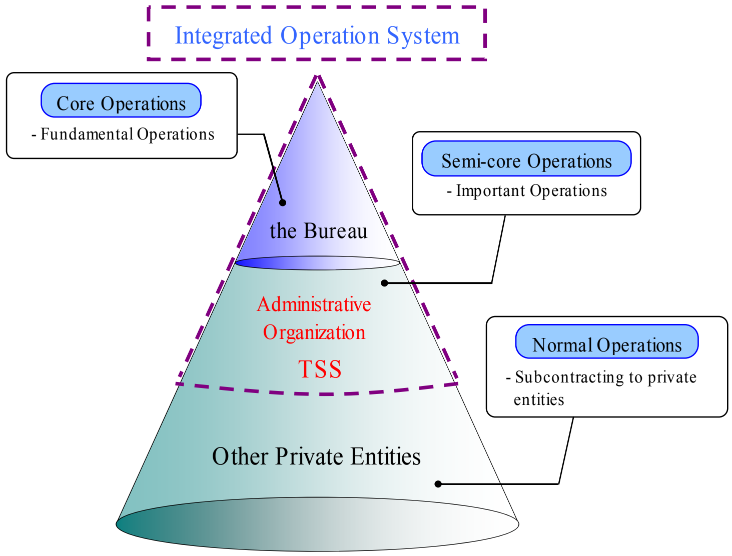


Figure 1. Conceptual Diagram of Operation System

3. INVESTIGATIONS AND DIAGNOSES OF WATER DISTRIBUTION PIPELINES

Investigations and diagnoses of water distribution pipelines of which TSS takes charge consist of three investigations as shown in Figure 2.

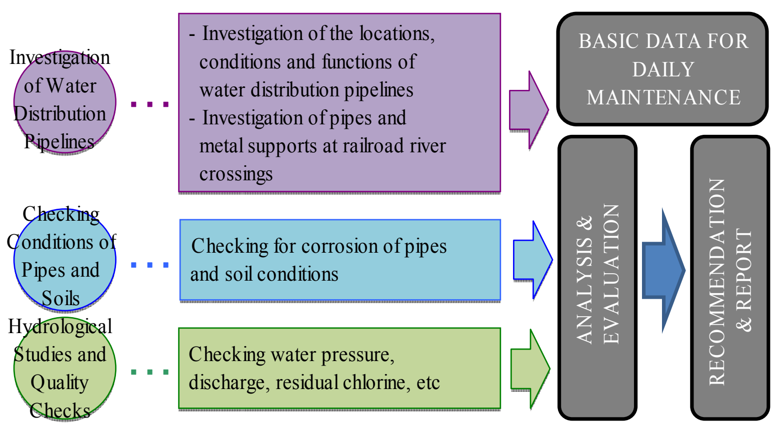


Figure 2. Investigations and Diagnoses of Water Distribution Pipelines

Among these investigations, the result of investigations of distribution pipes is useful basic data for daily maintenance as well as grasping the location and the condition of pipes and attached facilities such as valves, and has become essential for rapid restoration work.

Moreover, with the result of the check of conditions of pipes and surrounding soils, the analyses and evaluations about safety of pipes and corrosiveness of soil are scientifically executed, and on the basis of them the recommendation of improving distribution facilities are shown to the Bureau. The report of these results is utilized as a basic database by the Bureau for drawing up a pipe renewal plan and makes a great contribution to building up an earthquake-resistant pipeline network by earthquake proofing with replacement project of pipes.

TRANSITION OF MATERIALS USED FOR DISTRIBUTION PIPES FOR TOKYO WATERWORKS

At present the total length of water distribution pipes which is managed by the Bureau is 25,652km, and they are classified as shown in Table 1.

Table 1. Total Length of Distribution Pipes Managed in Tokyo Waterworks (as of the end of FY 2007)

<table>
<tr><th colspan="2">Kind</th><th>Distribution Mains
(ϕ400 - 2700)
(km)</th><th>Distribution Sub-mains
(ϕ50 - 350)
(km)</th><th colspan="2">Total</th><th colspan="2">Proportion
(%)</th></tr>
<tr><td rowspan="2">Aged Pipes</td><td>Cast-iron Pipes</td><td>30
(27)</td><td>197
(136)</td><td>227
(163)</td><td rowspan="2">244
(178)</td><td>0.9
(1.0)</td><td rowspan="2">1.0
(1.1)</td></tr>
<tr><td>Steel Pipes</td><td>14
(14)</td><td>3
(1)</td><td>17
(15)</td><td>0.1
(0.1)</td></tr>
<tr><td colspan="2">Early Ductile Iron Pipe Routes</td><td>163
(143)</td><td>2,485
(1,771)</td><td colspan="2">2,648
(1,914)</td><td colspan="2">10.3
(12.0)</td></tr>
<tr><td colspan="2">Ductile Iron Pipes</td><td>1,871
(1,285)</td><td>20,527
(12,344)</td><td colspan="2">22,398
(13,629)</td><td colspan="2">87.3
(85.3)</td></tr>
<tr><td colspan="2">Steel Pipes</td><td>227
(203)</td><td>122
(62)</td><td colspan="2">349
(265)</td><td colspan="2">1.4
(1.7)</td></tr>
<tr><td colspan="2">Others</td><td>0
(0)</td><td>13
(0)</td><td colspan="2">13
(0)</td><td colspan="2">0.1
(0)</td></tr>
<tr><td colspan="2">Total</td><td>2,305
(1,672)</td><td>23,347
(14,314)</td><td colspan="2">25,652
(15,986)</td><td colspan="2">100
(100)</td></tr>
</table>

( ) shows data in the ward area of Tokyo, included as for managed length.

As for the materials of water distribution pipes for Tokyo waterworks, cast-iron pipes had been mainly adopted since1898, the beginning of service. These pipes are called “Aged Pipes.”

After ductile iron pipes appeared as reinforced iron pipes and became available, the Bureau began to switch the applicable kind of pipes to ductile iron pipes gradually from around 1960.

Many ductile iron pipelines which have been installed by FY 1972 are called “Early Ductile Iron Pipelines” and they are distinguished from others because conventional cast-iron pipes have been mixed for special fittings such as bend pipes on them.

1. AGED PIPES

The types which consist of aged pipes are mainly cast-iron pipes. They have much possibility of water leakage because of damage with their low strength, and rusty water since mortar lining is not executed inside them. Nowadays, since the length of aged pipes is 244km, the Bureau is diligently conducting the replacement of aged pipes.

2. EARLY DUCTILE IRON PIPELINES

The period of installing early ductile iron pipelines corresponds to the transition period from cast-iron pipes to ductile iron pipes, and nowadays, the length of early ductile iron pipes is 2,648km.

Figure 3 shows the sequences of adopting ductile iron pipes to water distribution pipes. The specifications of ductile iron pipes have been revised in respective diameters or periods, and the characteristics are following:

- Ductile iron pipes have been adopted only for straight parts of water distribution network from the beginning, meanwhile most special fittings are made of conventional cast-iron with lower strength and lower earthquake-resistance.
- In earthquakes pipes can slip off as no restraining function belongs to the joint of them.
- As mortar lining for corrosion-proof has not been executed inside the ductile iron pipes which have been adopted for several years since the beginning, their terrible corrosion causes rusty water.
- As they are not covered with polyethylene sleeve for corrosion-proof, the corrosion of the outside of pipes and bolts is deteriorating and can cause water leakage and slipping off of pipes in earthquakes.

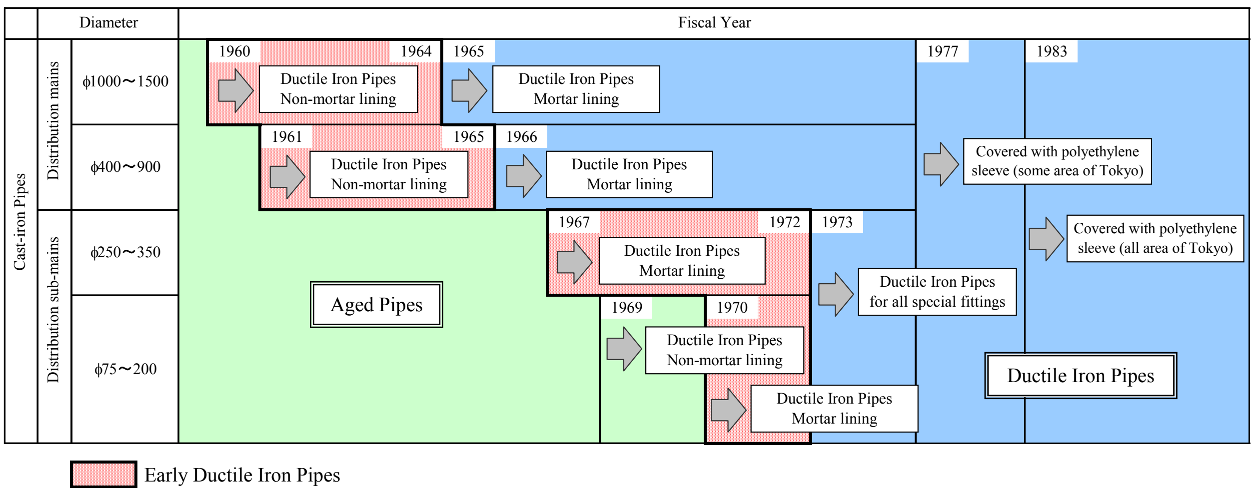


Figure 3. Circumstances of Adopting Ductile Iron Pipes to Water Distribution Pipes

As mentioned above, the network which ductile iron pipes have been installed by early 1970's contains some types of incomplete pipes with such many problems on maintenance as water leakages or rusty water caused by corrosion inside/outside pipes, and risk of earthquake damages because of weakened pipes with decrease of pipe thickness and lack of restraint fittings.

INVESTIGATIONS AND ANALYSES OF EARLY DUCTILE IRON PIPES

The Bureau decided to investigate the degrading of pipes, surrounding soil, and so on using experimental construction sites of replacing pipes from FY 2001 to FY 2003 to solve maintenance issues on early ductile iron pipes. The investigations were entrusted to TSS because of its long years of abundant achievements about investigations and diagnoses of water distribution pipelines.

TSS analyzed the result of the investigations to prepare a basic database in deciding priority of replacing early ductile iron pipes in the distribution pipelines.

This chapter makes a report of investigations and analyses of early ductile iron pipes conducted by TSS.

1. CONTENTS OF INVESTIGATIONS

The subjects of investigations are distribution mains of ductile iron installed from FY 1960 to FY 1965 (some of special fittings are made of cast-iron), and distribution sub-mains of ductile iron installed from FY 1967 to FY 1972 (special fittings are made of cast-iron), without mortar lining inside their special fittings.

The respective investigating items were as shown on Table 2, and the condition of corrosion was quantitatively, the condition of soil was physicochemically investigated.

Table 2. Items of Investigations

Kind		Items
Pipe Condition Investigation	Outside	Materials & Type of Joints, Condition of Corrosion on Pipes & Bolts, Measurement of the Depth of Corrosion (Pitting), Measurement of Pipe Thickness, etc.
	Inside	Measurement of Neutralized Mortar Lining Thickness, Measurement of Scale Thickness (Clogging Rate), etc.
Soil Investigation	Field Survey	Measurement of Evaluation Items for Corrosive Soil by ANSI such as Specific Resistance, pH, Redox Potential, Moisture & Sulfide
	Laboratory Test	Measurement of Moisture Content, Sulfide Content, Chloride Ion Content, Sulfate Ion Content, Amount of Evaporated Residue & Specific Resistance of Extracted Water

As for investigation points, per 100 meters of replacing length, generally one for straight pipes, and one and over for special fittings were investigated on construction sites of replacing pipes in the ward area of Tokyo. The number of points is shown on Table 3.

Table 3. Number of Investigation Point

Kind	Pipe Condition Investigation (pts.)			Soil Investigation (pts.)
	Straight Pipes	Special Fittings	Total	
Distribution Mains (ϕ400 and over)	137	147	284	73
Distribution Sub-mains (ϕ350 or under)	843	1,356	2,199	603
Total	980	1,503	2,483	676

2. METHOD OF INVESTIGATIONS

(1) Pipe Condition Investigation

1) Materials and type of joints (Judged from observations)
2) Condition of corrosion on pipes and bolts (Judged from observations)
3) Measurement of the depth of corrosion (Pitting) (The depth of the pitting found by removing soil or others)
4) Measurement of pipe thickness (4 points of top, bottom, left and right)
5) Measurement of neutralization of mortar lining on straight pipes (The mortar thickness of the point where the neutralization is progressing the most among 4 areas of top, bottom, left and right)
6) Measurement of scale thickness (Clogging rate) (The scale thickness of the point where the most scale sticks among 4 areas of top, bottom, left and right)

(2) Soil Investigation

1) Field survey (According to evaluation items for corrosive soil by ANSI*)

- Moisture; presence of groundwater and condition of dampness
- Specific resistance; measured on 4 points of top, bottom, left and right
- pH, sulfide and redox potential; measured by taking soil test pieces from 4 points of top, bottom, left and right around target pipes

*ANSI = American National Standards Institute

2) Laboratory test (6 items such as chloride ion content are measured in an analyzing facility by taking soil test pieces from 2 points of top and bottom around target pipes)

3. RESULT OF PIPE CONDITION INVESTIGATION

(1) Materials and Type of Joints (Table 4, Figure 4)

Table 4. Materials and Type of Joints of Investigated Pipes

Kind		Cast-iron (pts.)			Ductile Iron (pts.)	
		Socket Type	A Type	Flange Type	A Type	T Type
Distribution Mains	Straight Pipes	0	0	0	137	0
	Special Fittings	1	62	0	86	0
Distribution Sub-mains	Straight Pipes	0	0	0	818	25
	Special Fittings	0	1,353	1	0	1

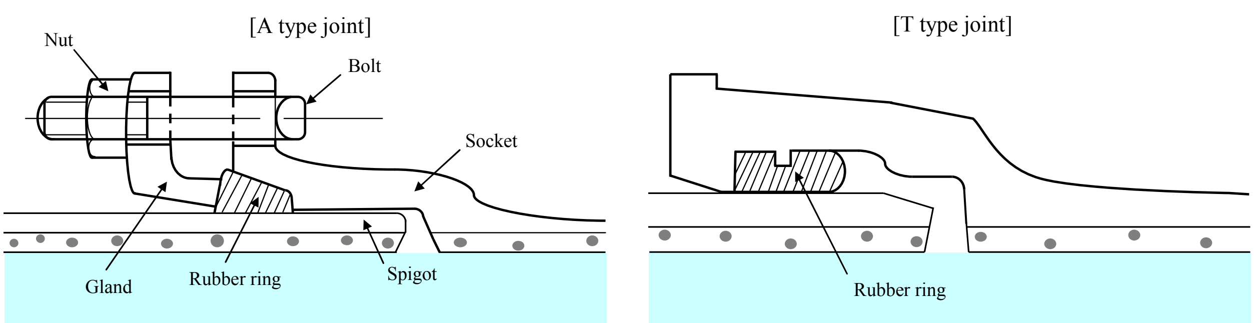


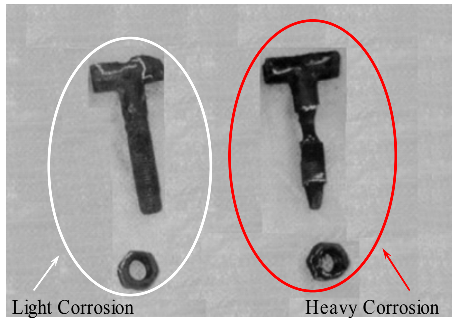
Figure 4. Structure of Joints

Nearly half of special fittings of water distribution mains and special fittings of water distribution sub-mains are made of cast-iron. And almost all of the types of joints were A type or T type which were not restraint fittings.

(2) Corroding Condition of Pipes (Outside) and Bolts (Picture 1)

As for the corrosion condition of the pipes (outside), at more than half of the investigating points, corrosion of more than medium degree was observed. Rust was growing extensively on the part where the painting fell off because of aging and degrading.

At nearly half of the investigating points the bolts corroded seriously. The loss of bolt bodies was extensively found at the thread of screws and many bolts degraded to the shape like sharpened pencils from tips to fastened nuts.



Picture 1. Corrosion Condition of Bolts

(3) Depth of Corrosion (Pitting) (Figure 5, Picture 2)

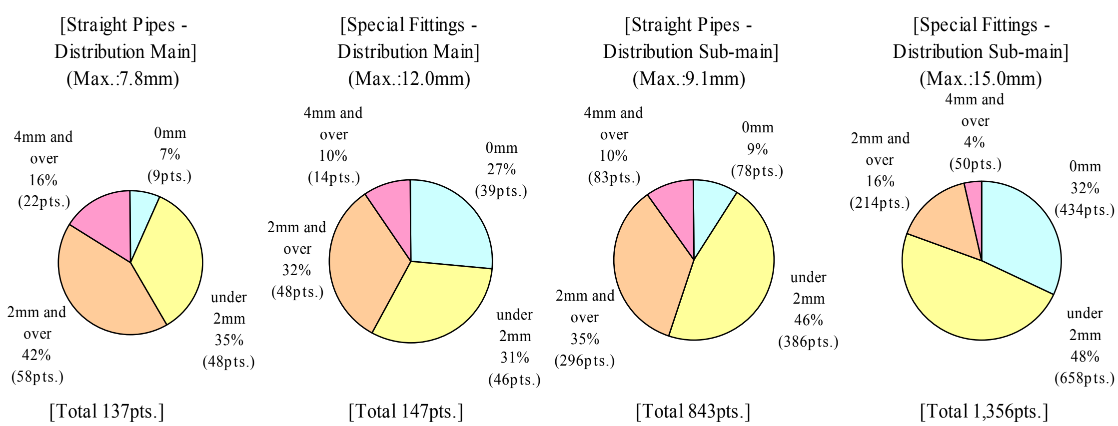
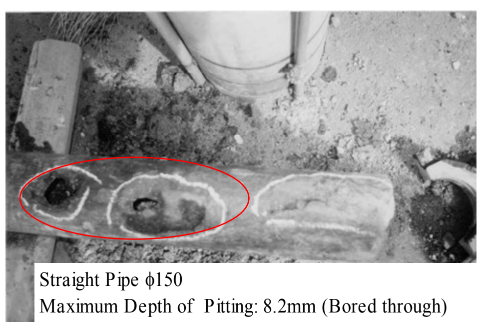


Figure 5. Depth of Corrosion (Pitting) outside Pipes



Picture 2. Condition of Corrosion (Pitting) outside Pipes

The maximum depth of pitting of special fittings was larger than that of straight pipes, but pittings were observed on more than half of the respective pipes. On some straight pipes of small pipe thickness on water distribution sub-mains, it was observed that iron body was bored through by pitting and only mortar lining prevented water leakage.

(4) Pipe Thickness (Figure 6)

As for straight pipes from ϕ100 to 400 of which standard thickness is 7.5 mm, few of them became very thin though it has been passed about 40 years since they were installed, and the dispersion of thickness was not much (the 4 points -top, bottom, left and right- of section to be investigated were freely determined to the axis direction of pipes regardless of the corroding condition).

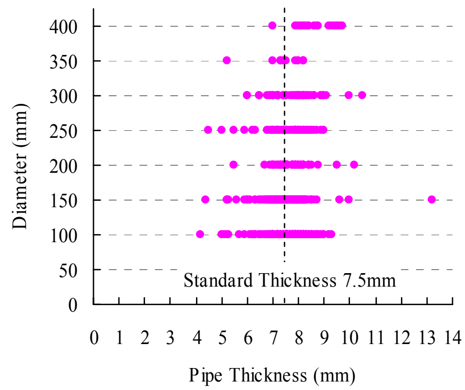


Figure 6. Pipe Thickness from ϕ100 to 400

(5) Neutralization of Mortar Lining

As for the neutralization rate (maximum thickness of neutralized mortar lining / maximum thickness of mortar lining) of both water distribution mains and sub-mains, more than half of them accounted for over 50% and the progression of neutralization was observed, but no mortar was falling off.

(6) Thickness of Scale (Clogging Rate) (Figure 7, Picture 3)

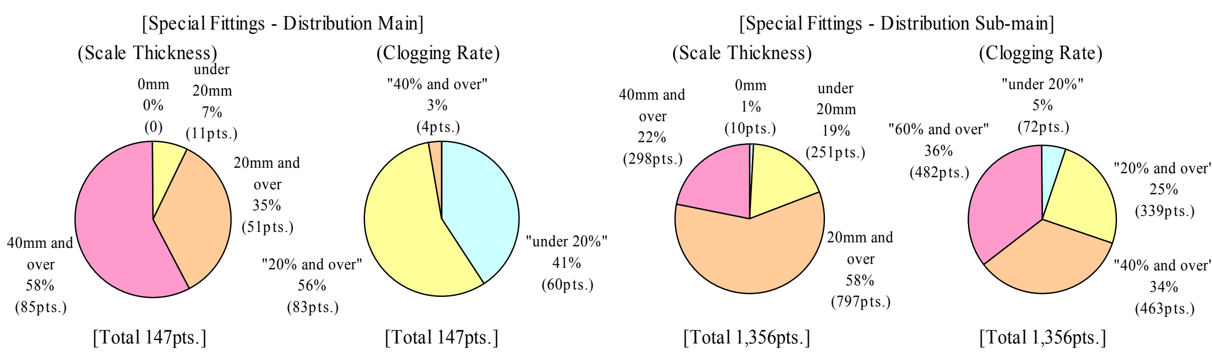
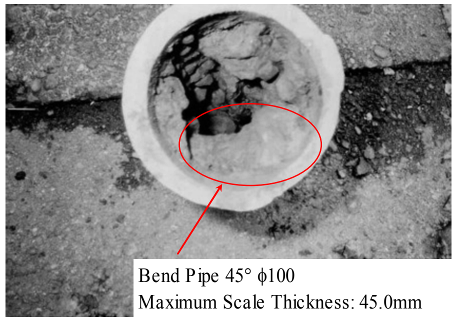


Figure 7. Scale Thickness and Clogging Rate of Pipes

As for the scale inside special fittings, 20mm thickness and more were found on 93% of water distribution mains, 80% of sub-mains. As for the clogging rate (valid cross section taking account of scale thickness calculated with averaging 4 points -top, bottom, left and right- / whole cross section deducting scale thickness), 40% clogging rate and over were found on 70% of pipes. As a clogging rate of a pipe amounted to 99%, the flow was almost impossible.



Picture 3. Sticking Condition of Scale inside a Pipe

4. EARTHQUAKE-RESISTANCE OF PIPELINES

The straight pipes on early ductile iron pipelines are made of ductile iron and have more strength and earthquake-resistance than cast-iron ones. But Figure 8 shows that they are with shallow insertion and without restraining functions, so in earthquakes the damage caused by the slipping off of pipes like at South Hyogo Prefecture Earthquake in 1995 will be predicted.

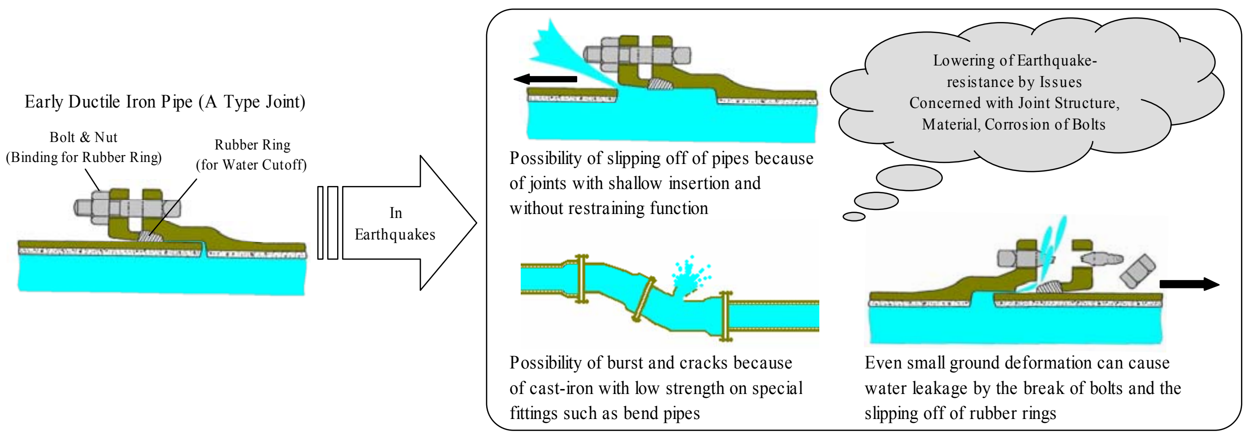


Figure 8. Earthquake-resistance of Early Ductile Iron Pipelines

In addition to this, special fittings are mostly made of cast-iron, with low strength and low earthquake-resistance. Therefore the earthquake-resistance of early ductile iron pipelines is comprehensively low.

Moreover, as it became clear that nearly half of bolts were seriously corroding with losses or thinning, these corrosions can cause the break of bolts and the slipping off of rubber rings in earthquakes. In particular, at the area where liquefaction occurs easily, even relatively small earthquakes can cause damage.

Meanwhile as the thickness of pipes locally decreased because of corrosion inside/outside pipes, the stress on pipes could increase and reduce the safety in terms of pipe strength.

5. RESULT OF SOIL INVESTIGATION

(1) Field Survey

The depth of corrosion (pitting) outside pipes by intensity of corrosion based on the soil evaluation standard by ANSI of Table 5 and the corroding condition of bolts are as shown in Figure 9 and Figure 10. The tendency was found that the depth of pitting and corrosion of bolts were to heavier degree as the corrosiveness was more remarkable.

Table 5. Score Chart and Evaluation Standard of Soil Evaluation Method by ANSI

Items	Measured Value	Point	Items	Measured Value	Point
Specific Resistance (kΩ-cm)	< 1.5	10	Redox Potential (mV)	> 100	0
	1.5 - 1.79	8		51 - 100	3.5
	1.8 - 2.09	5		0 - 50	4
	2.1 - 2.49	2		< 0	5
	2.5 - 2.99	1	Moisture	Much	2
	> 3.0	0		A Little	1
pH	< 2	5		Dry	0
	2 - 4	3	Sulfide	Existent	3.5
	4.1 - 6.5	0		A Little	2
	6.6 - 7.5	0*		No	0
	7.6 - 8.5	0	Extract from ANSI A21.5-1999		
	> 8.5	3			

* 3 points are added in the presence of sulfide and in low Redox potential.

Evaluation Standard

Corrosiveness	Light	Medium	Heavy
Point	1.0 - 7.0	7.5 - 9.5	10.0 and over

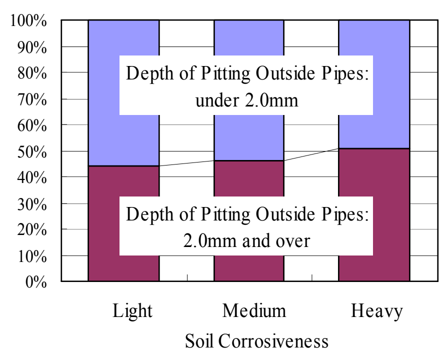


Figure 9. Soil Evaluation Based on Standard by ANSI and Corrosion outside Pipes

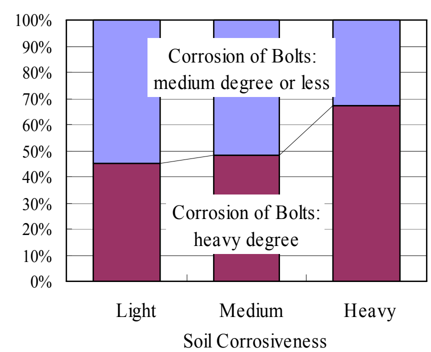


Figure 10. Soil Evaluation Based on Standard by ANSI and Corrosion of Bolts

(2) Laboratory Test

The result of evaluation based on the evaluation standard of soil analysis in Table 6 is as shown in Figure 11.

6. DIAGNOSIS OF CORROSION RATE OF EARLY DUCTILE IRON PIPES

Based on the acquired results of investigations, according to the procedure of Figure 12 by analyzing causality between corrosion (pitting) outside pipes and installing circumstances, and by preparing corrosion estimation formula, the number of years till pipes come up to the established degradation rate rank was calculated.

Table 6. Evaluation Standard of Soil Analysis

Items	Corrosiveness		
	Light	Medium	Heavy
Moisture Content (%)	under 20	20 and over	-
Sulfide Content (%)	under 0.1	0.1 and over under 0.3	0.3 and over
Chloride Ion Content (mg/l)	under 10	10 and over under 50	50 and over
Sulfate Ion Content (mg/l)	under 100	100 and over under 200	200 and over
Amount of Evaporated Residue (mg/l)	under 250	250 and over under 500	500 and over
Specific Resistance of Extracted Water (Ω-cm)	1,500 and over	-	under 1,500

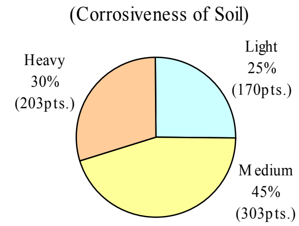


Figure 11. Result of Evaluation Based on Evaluation Standard of Soil Analysis

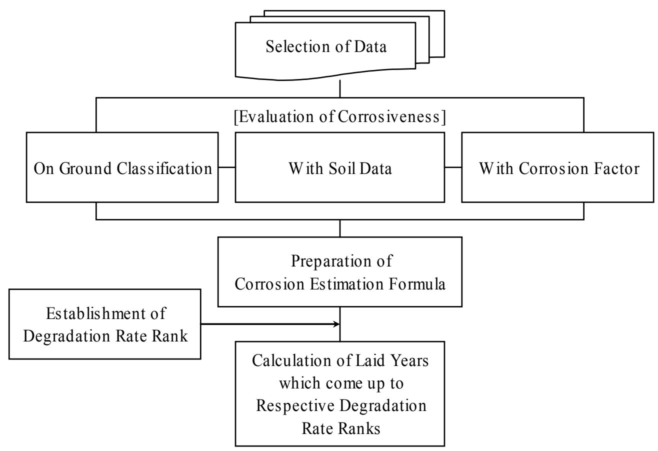


Figure 12. Flow for Diagnosis of Corrosion Rate

(1) Selection of Data (Number of data: 1,531)

- Investigation of early ductile iron pipes (2001-2003): 866 data
- Investigation of pipe and soil condition (1987-2002): 665 data

Condition for selecting of subjects:

- The material of a pipe is ductile iron
- The maximum depth of pitting is 0.5mm and over
- Polyethylene sleeve for corrosion-proof outside pipes is not executed

(2) Evaluation of Corrosiveness by Estimation Model Formula ($\eta = kt^{\alpha}$)

- η: predicted depth of pitting (mm), k: evaluation coefficient for corrosiveness, t: laid years, α: constant
  k is calculated from laid year and the depth of pitting which are acquired by investigations. α is 0.4 in the case of ductile iron pipe[1].
- By evaluating the corrosiveness with the estimation model formula, 12 kinds of ground in the ward area of Tokyo were classified into Ground A, B, C and D in the order of low

corrosiveness. After the statistical analyses, it was found that the soil of Ground C and D contains more clay and sulfide with high ANSI score.

(3) Preparation of Corrosion Estimation Formula (Table 7)

Table 7. Corrosion Estimation Formula

	Ground A	Ground B	Ground C	Ground D
Estimation Formula I	$\eta = 0.554 \times t^{0.566}$	$\eta = 0.631 \times t^{0.566}$	$\eta = 0.654 \times t^{0.566}$	$\eta = 0.725 \times t^{0.566}$
Estimation Formula II	$\eta = 0.562 \times t^{0.581}$	$\eta = 0.561 \times t^{0.581}$	$\eta = 0.627 \times t^{0.581}$	$\eta = 0.657 \times t^{0.581}$

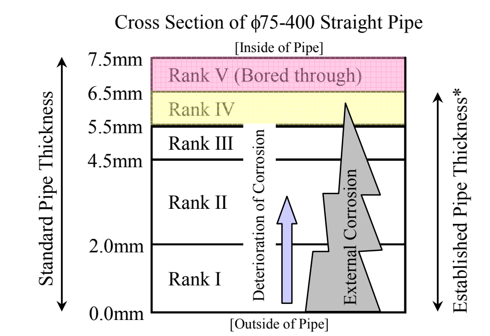
With the estimation model formula, the estimation formula I of which explanatory variable is Ground, and the estimation formula II of which explanatory variable is sulfide and corrosion factor of ANSI score, were prepared by the multiple linear regression analysis.

(4) Establishment of Degradation Rate Rank

The standard of evaluation at which stage the danger of water leakage appears according to estimated depth of pitting (degradation rate rank) was established as shown in Table 8. The evaluation of water leakage risk is by diameter since standard thicknesses of pipes vary by diameter. As an example, an image of the degradation rate rank for ϕ75-400 straight pipes is shown in Figure 13.

Table 8. Definitions and Countermeasures for Degradation Rate Rank

Degradation Rate Rank		Definition and Countermeasure
V	Definition	Bored through
	Countermeasure	Urgent countermeasure such as immediate replacement is needed because remaining pipe thickness is not basically guaranteed
IV	Definition	Less than 1.0mm of remaining pipe thickness until corrosive boring
	Countermeasure	Urgent replacement is needed (at latest by the progression of degradation rate to rank V) because the danger of water leakage is increasing
III	Definition	1.0mm and over, and less than 2.0mm of remaining pipe thickness until corrosive boring
	Countermeasure	Because corrosion is deteriorating, the replacement plan should be prepared for some pipes considering their importance
II	Definition	Depth of pitting is over 2.0mm reserved for corrosion
	Countermeasure	Investigations or diagnoses are needed within 20 years as duration of this rank because progress of pitting is predicted
I	Definition	Depth of pitting is much less than 2.0mm reserved for corrosion
	Countermeasure	No particular countermeasure is needed as there is enough reserve for corrosion



* Established Pipe Thickness = Standard Thickness - Permissive Dispersion
(Permissive Dispersion of Pipe Thickness: -1.0mm for pipes of which standard thickness is 10mm or under, -10% of standard thickness for pipes of which standard thickness is over 10mm)

Figure 13. Image of Degradation rate Rank

(5) Calculation of Laid Years which Come up to Respective Degradation Rate Ranks

With the estimation formula I and II, the laid years which come up to the degradation rate rank IV and V were calculated. Based on “(2) Evaluation of Corrosiveness by Estimation Model Formula, ”Ground A and B which range in the western ward area of Tokyo were roughly classified into Ground I, and Ground C and D which range in the eastern ward area of Tokyo were roughly classified into Ground II(Figure 14). The result of estimated laid years which come up to the dangerous stage of water leakage was as shown in Table 9.

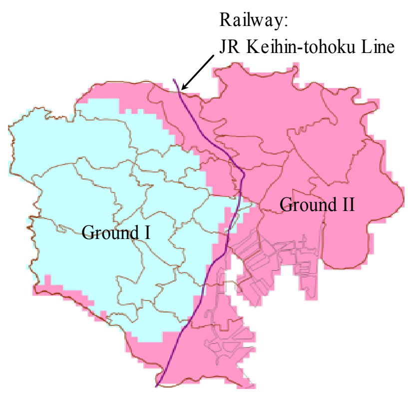


Figure 14. Classification of Ground in the Ward Area of Tokyo

Table 9. Estimation of Laid Years which Come up to Dangerous Stage of Water Leakage

Diameter (mm)	Ground I		Ground II	
	Degradation Rate Rank IV	Degradation Rate Rank V	Degradation Rate Rank IV	Degradation Rate Rank V
75 - 400	48 years	65 years	39 years	52 years
500	65 years	82 years	52 years	67 years
600	93 years	over 100 years	75 years	92 years
700	over 100 years	over 100 years	90 years	over 100 years
800 and over	over 100 years	over 100 years	over 100 years	over 100 years

THE REPLACEMENT PROJECT OF PIPES BASED ON THE INVESTIGATIONS AND THE ANALYSES

The Bureau evaluated the result of the investigations and the analyses about early ductile iron pipes, and set to the replacement project of early ductile iron pipes in FY 2005, with establishing the priority of replacing pipes and the target achievement period.

1. EVALUATION OF THE INVESTIGATIONS AND THE RESULT OF ANALYSES

The evaluation of early ductile iron pipes is as follows:

- Earthquake Resistance for the structure of joints and the material of special fittings is low.
- The corroding bolts tend to break in earthquakes and so on. In particular, the Ground II area has much possibility of liquefactions as shown in Figure 15 with a risk of damage even in small earthquakes.
- Ground II area where the external corrosion deteriorates rapidly, has much possibility of water leakage for thin pipes of under ϕ400 diameter.

- The neutralization of mortar lining and scale inside pipes cause water quality degrading, rusty water and decrease of flow capacity.

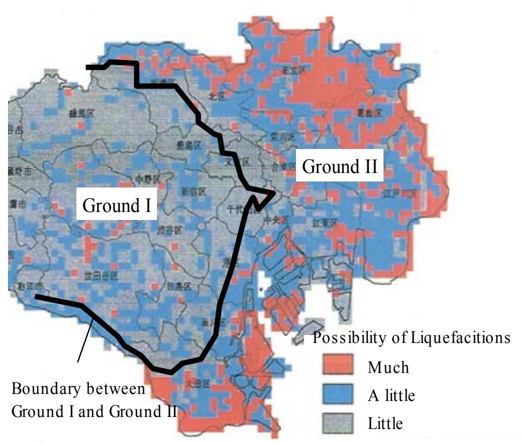


Figure 15. Map for Judging Liquefaction of Ground in the Ward Area of Tokyo[2]

2. THE BASIC POLICY OF REPLACEMENT

By the comprehensive evaluation about the degrading of pipes, the quantitative analysis of the degrading situation as shown in Table 10 was conducted and the basic policy for the help of deciding the priority of replacing pipes was stated.

Table 10. The Quantitative Evaluation for Degrading Situation of Early Ductile Iron Pipes

Classification of Routes	Earthquake-resistance of Pipelines		Possibility of Water Leakage by External Corrosion in Future		Rusty Water and Decrease of Flow Capacity by Inside Degradation		Influence of Accidents such as Water Leakage
	Ground I	Ground II	Ground I	Ground II	Ground I	Ground II	
Distribution Sub-mains Normal Routes*2	High Risk - Without Restraining Functions of Joints - Special Fittings are made of Cast-iron - Corrosion on Bolts - Non-liquefactions	High Risk - Without Restraining Functions of Joints - Special Fittings are made of Cast-iron - Corrosion on Bolts - Liquefactions	Intermediate Risk - Thin Pipes - Slow External Corrosion	High Risk - Thin Pipes - Rapid External Corrosion	High Risk - Much Possibility of Rusty Water - Rapid Decrease of Flow Capacity by Scale		Medium
Distribution Sub-mains Important Routes*1							Large
Distribution Mains Normal Routes	Intermediate Risk - Without Restraining Functions of Joints - Some Special Fittings are made of Cast-iron - Corrosion on Bolts - Non-liquefactions	High Risk - Without Restraining Functions of Joints - Some Special Fittings are made of Cast-iron - Corrosion on Bolts - Liquefactions	Low Risk - Thick Pipes - Slow External Corrosion	Low Risk - Thick Pipes - Rapid External Corrosion	Intermediate Risk - Much Possibility of Rusty Water - Slow Decreasing of Flow Capacity by Scale because of Large Diameter		Large
Distribution Mains Important Routes							Very Large

Quantified

[Risk] High: 4 or 5 pts., Intermediate: 3 pts., Low: 1 or 2 pts.
[Influence] Medium: ×3, Large: ×4 or 5, Very Large: ×7

Classification of Routes	Earthquake-resistance of Pipelines		Possibility of Water Leakage by External Corrosion in Future		Rusty Water and Decrease of Flow Capacity by Inside Degradation		Influence of Accidents such as Water Leakage	Total Points	
	Ground I	Ground II	Ground I	Ground II	Ground I	Ground II		Ground I	Ground II
Distribution Sub-mains Normal Routes	4	5	3	5	5	5	×3	36	45
Distribution Sub-mains Important Routes	4	5	3	5	5	5	×4	48	60
Distribution Mains Normal Routes	3	4	1	2	3	3	×5	35	45
Distribution Mains Important Routes	3	4	1	2	3	3	×7	49	63

*1 Important Routes are on the viewpoint of water supply and control main transmission/distribution pipelines forming the framework of pipeline network, and supply water to such important facilities in earthquakes as emergency water tanks, places of refuge and medical institutions.
*2 Normal Routes are except Important Routes.

(1) Priority

The important routes mean ones on that countermeasures for earthquakes shall be taken in maximum priority since the influence of water suspension and water leakage ranges widely. Therefore starting replacing pipes from the important routes is the most effective.

As for normal routes, the corrosion inside pipes seems to range in both Ground I and II without regional tendency, but the tendency of external corrosion is divided into Ground I area with low deterioration and Ground II area with high deterioration. And in Ground II area there is possibility of the slipping off of joints and water leakage by the corrosion of bolts from the fact that Ground II area almost entirely corresponded to the area with much possibility of liquefaction in earthquakes. Consequently, in replacing normal routes, Ground II area should be given precedence from viewpoints of countermeasures of earthquakes and water leakage.

(2) Target Achievement Period

It was predicted that the risk of water distribution sub-mains in Ground II area would come up to the dangerous level in around FY 2019 (52 years after the adoption of early ductile iron pipelines). In Ground I area, the deterioration of corrosion was predicted to be around 10 or 15 years slower than that in Ground II area. The result of quantitative evaluations showed that, as for the laid years to the dangerous level, that of water distribution mains is longer than that of water distribution sub-mains, but as the influence range of mains is larger than that of sub-mains, the respective priorities are almost the same in the end.

3. THE IMPLEMENTATION POLICY OF REPLACEMENT

Based on the fundamental viewpoint for the replacement, the replacement project has been set to work under the following implementation policy.

- From FY 2005 to FY 2018 on both water distribution mains and sub-mains in principle the replacements of the following routes which are older and have more possibility of water leakage are given priority.
  - Important routes (about 500 km in the ward area of Tokyo)
  - Normal routes in remarkably corrosive soil area (Ground II) (about 700 km in the ward area of Tokyo)
- From FY 2007 to FY 2021 the replacement of early ductile iron pipelines (excluding the above-mentioned pipelines) shall be conducted (about 920 km in the ward area of Tokyo). By the way, as for Tama area (which lies west of the ward area in Tokyo), since the nature of soil is the same as that of Ground I in the ward area of Tokyo, about 740 km of replacement shall be conducted by FY 2021, as the important routes are given priority.

4. THE EFFECTS OF REPLACEMENT

The result of investigation or others, it is found that the replacement gives effects as follows:

1) Joints

By replacing to new pipes with restraint fittings, the slipping off of pipes can be prevented and the functions of pipelines are maintained even in earthquakes.

2) The Strength of Pipes

All the pipes including special fittings shall be replaced to ductile iron pipes with more strength. Moreover, such countermeasures for the corrosion-proofing as mortar lining inside pipes and polyethylene sleeves to cover pipes, can secure the necessary thickness of pipes and the strength of bolts sustainably. These enable to maintain the strength of pipes which can withstand earthquakes.

The replacement project which has been started in FY 2005 is getting rid of early ductile iron pipelines steadily as well as building up the earthquake-resistant distribution network as shown in Figure 16. Furthermore, with conducting countermeasures of corrosion-proofing inside/outside pipes, keeping low leakage rate from now on (Figure 17) and the prevention of rusty water in advance will be also expected.

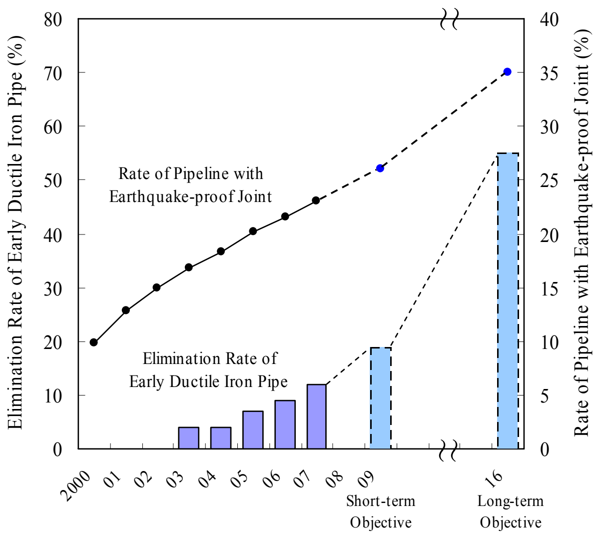


Figure 16. Transition of Indices Concerned with Facility Development brought by Replacing Pipes

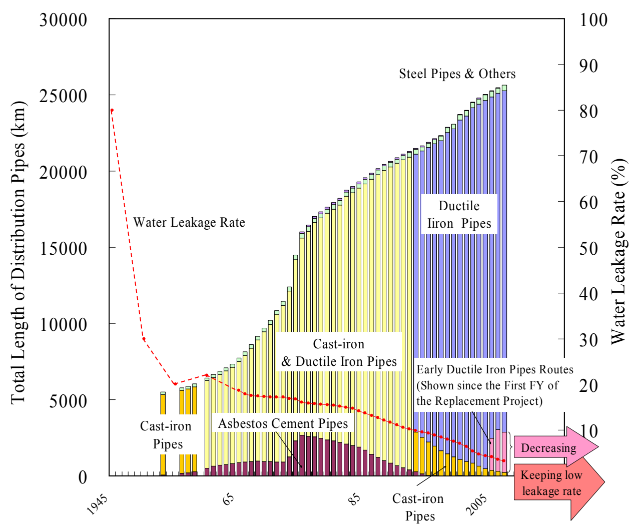


Figure 17. Transition of Total Length of Distribution Pipes and Water Leakage Rate

CONCLUSIONS

The appropriate management of water pipelines is essential for stable and sustainable water supply. Collecting and analyzing basic data are needed with priority for securing the budget and scheduled management.

The FY 1987, when TSS started the work, was the period when the rate of service pervasion of Tokyo waterworks came closer to nearly 100% and maintenance of facilities began to be given importance.

In response to this tendency, the continuous investigations for pipes and collecting numerous basic data such as the conditions of constructing, corroding and the circumstances of installation resulted in making issues of early ductile iron pipelines stand out.

Then, by narrowing down the subject of 3 years of investigations and analyses to early ductile iron pipelines, the replacement project could be conducted effectively with priority.

While the replacement project can improve the earthquake-resistance of pipelines and the prevention of water leakage, appealing for the necessity of replacing pipelines on a quantitative viewpoint could made various people concerned understand the effect smoothly and enabled to start the project.

Finally, it shall be recognized that the integrated implementation of respective roles of the Bureau and TSS enabled to solve issues efficiently.

THANKS

In accomplishing this paper, the Bureau made various supports such as providing data. We would like to express gratitude here.

REFERENCES

[1] Yukio Katano, Haruki Morikawa, and Yoichi Igawa, Corrosion Prediction for pipes in Aggressive Soils, KUBOTA TECHNICAL REPORT No.23, pp.88-95, 1991.
[2] Tokyo Disaster Prevention Council, The Research about the Prediction of Damage by Earthquakes in Tokyo, Sep.1991.

Seismic Response of San Pablo Reservoir Drain Intake

Ahmed Nisar, David Lee, Nikolay Doumbalski and Eric Fieberling

ABSTRACT

The paper describes seismic analysis of the low-level outlet intake riser of the East Bay Municipal Utility District's San Pablo reservoir. The San Pablo reservoir, located two miles from the Hayward Fault, is a critical element of the District's system and provides approximately 30 million gallons of raw water each day for approximately 1.3 million customers.

The analysis was performed to help the District decide between the size and type of a new valve to replace a failed 66-inch butterfly valve. The failed valve was located in a nearly inaccessible chamber 140 feet below the reservoir surface. To make the valve more accessible, the District decided to install a new valve on top of the intake riser of the low-level outlet rather than replacing the failed butterfly valve in the existing valve chamber. The low-level outlet intake riser, a 78-inch diameter cement mortar coated and lined steel pipe that extends vertically 30 feet above its concrete foundation, was not originally designed to carry the additional load from the valve. Seismic assessment of the riser with the additional weight of the new valve and the expected earthquake shaking was required prior to placing the procurement order for the custom made valve, a significant capital cost item requiring Board approval.

A three dimensional nonlinear time history analysis of the riser and new valve was performed to assess the adequacy of the existing riser. The finite element model included soil-structure interaction effects and hydrodynamic loads from water, and was completed within an ambitious three week period to meet the overall schedule consisting of Board approval, valve procurement and a narrow 9-week construction window. Completion of work within this schedule was critical to have a functional low-level outlet prior to start of construction for the $60 million seismic improvement work on the 170 feet high and 1,200 feet long earthfill dam.

Details on the failure of the butterfly valve, key features of the new valve, and construction challenges such as installation restrictions, complications of underwater confined-space demolition, low-visibility valve installation, and the challenging project schedule are provided in a companion paper by Todaro and Wong (2008)[1].

Ahmed Nisar, Associate, MMI Engineering, 475 14th Street, Suite 400, Oakland, CA, 94612
David Lee, Senior Civil Engineer, EBMUD, 375 11th Street, Oakland, CA 94607
Nikolay Doumbalski, Project Professional, MMI Engineering, 475 14th Street, Suite 400, Oakland, CA, 94612
Eric Fiedberling, Associate Civil Engineer, EBMUD, 375 11th Street, Oakland CA 94607

INTRODUCTION

The San Pablo dam is a 170-foot high and 1,200-foot long earthfill embankment located approximately 8 miles from downtown Oakland. The dam impounds a 38,600 acre-foot reservoir that provides approximately 30 million gallons of water per day to 1.3 million customers, and is a critical element of the East Bay Municipal Utility District (District) water system.

The San Pablo dam is located approximately 2 miles east of the seismically active Hayward fault (**Figure 1**). In the San Francisco Bay Area, the Hayward fault is one of the most significant features of the San Andreas Fault system. The Working Group on California Earthquake Probabilities, a group of leading scientists, academicians, researchers, and practitioners have estimated a 31% probability of an earthquake of Magnitude 6.7 or greater on the Hayward fault in the next 30 years.[2]

An assessment of the seismic stability of the dam embankment showed the potential for significant deformation under a Magnitude 7.25 scenario earthquake on the Hayward fault, which prompted the District to embark on a $60 million seismic retrofit project to improve the embankment stability. Prior to start of construction, it was necessary for the reservoir to have an operational low-level outlet to allow the reservoir to be drained, if needed. The 60-inch butterfly valve controlling the low-level outlet had been installed in a confined-space chamber when the reservoir was drained in 1980. After the reservoir was filled the valve became nearly inaccessible and when, during testing in 2006, the valve failed, it became necessary to replace it prior to the start of construction on the larger $60 million seismic retrofit project.

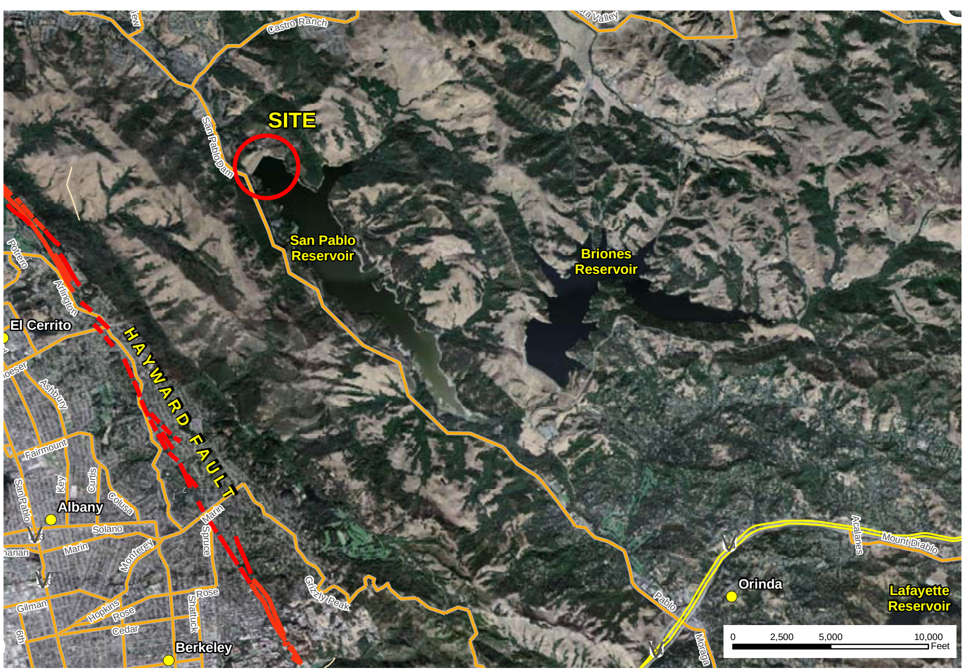


Figure 1: Site Location Map

The District, together with Lee C. Gerbig LLC, a large-valve consultant, prepared an alternatives evaluation report that recommended replacement of the failed butterfly valve with a knife gate valve located on top of the intake riser for the low level outlet system. The intake riser is a 78-inch diameter mortar lined and coated steel pipe that extends 30-feet vertically from the top of a concrete intake portal. This location was selected to improve access for maintenance and repair since the failed valve was located approximately 140 feet below the reservoir surface in an almost inaccessible chamber near the base of the outlet tower. Two different valve types (butterfly and knife gate) and two different valve sizes (60-inch and 78-inch) were considered The knife gate was the preferred option because unlike a butterfly valve, the knife gate disc operates directly from the cylinder without an intermediate mechanism, so it can be hydraulically isolated, detached, and brought to the surface for repair.

This paper describes the three dimensional nonlinear time history analysis performed to assess the seismic response of the intake riser with the additional weight of the new valve, since the riser had not been originally designed to carry the additional load from the valve, particularly under seismic loading. Two valve configurations, one assuming a 78-inch knife gate and the other a 60-inch butterfly valve, were considered to cover the four cases consisting of the two valve types and the two valve sizes. While the knife gate was the preferred option, the butterfly valve was evaluated because it was significantly lighter than the knife gate and it did not have the additional eccentric loading caused by the position of the hydraulic cylinder, the valve leaf, and the bonnet of the knife gate. Constraints associated with the construction schedule, procurement schedule, and Board approval for the capital expenditure to procure the custom valve required that the analysis was completed within a three week period.

LOW-LEVEL OUTLET SYSTEM DESCRIPTION

The low-level outlet is used exclusively to drain the reservoir. It is located near the western end of the embankment and east of the Sobrante outlet tower, as shown in **Figure 2**.

Figure 2: Low-Level Outlet Location

The low-level outlet system consists of an intake structure and a 600-foot long tunnel that connects to the base of the outlet tower, as shown in **Figure 3**.

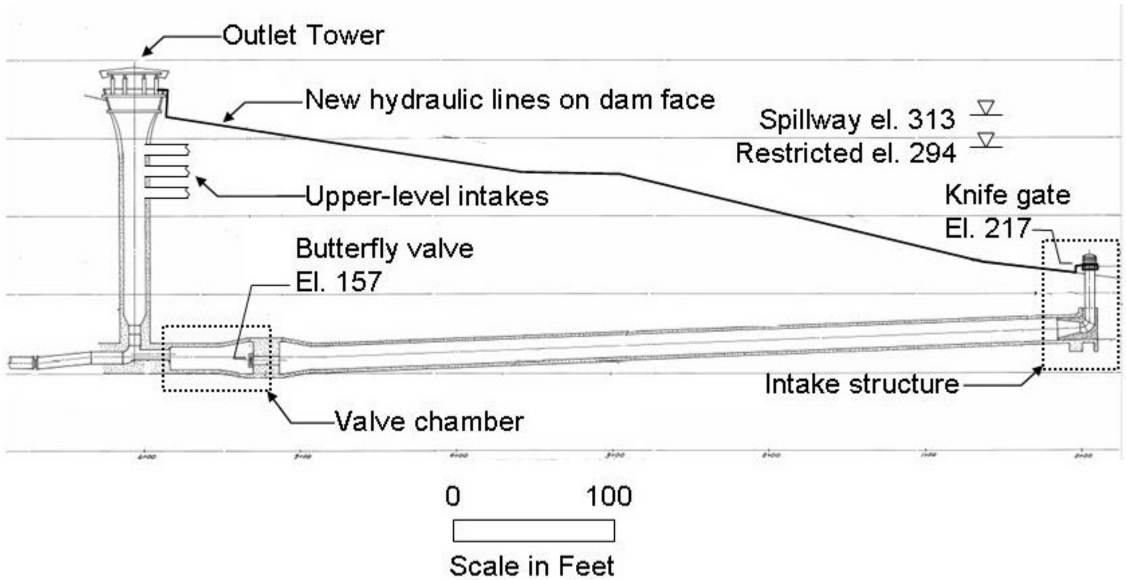


Figure 3: Low-level Outlet System

The intake pipe is a 78-inch inside diameter cement mortar lined and coated steel pipe extending 30 feet vertically as a cantilever from a concrete intake portal structure. At its base, the intake pipe is embedded in a 10-foot wide, 18.5-foot long and 17.6-foot high reinforced concrete block that forms part of the concrete intake portal structure. The intake portal structure has an "L" shaped cross section and is 25.25 feet by 18.5 feet in plan. The configuration of the intake portal structure is shown in **Figure 4**. At its top, the intake pipe has a trash rack supported on 94.5 inch (outside diameter) and 80.5 inch (inside diameter) ring flange. The trash rack is 6 feet tall and consists of 12 grating panels separated by 6-foot tall TS 6x4 columns. The top of the trash rack has a 72-inch diameter, 0.5-inch thick steel dished head.

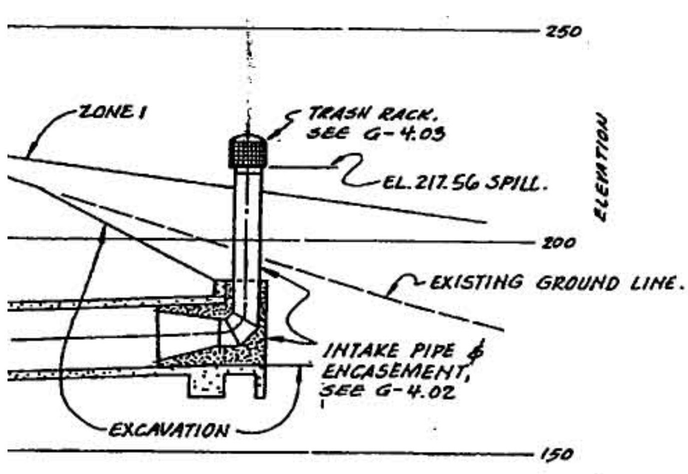


Figure 4: Intake pipe and portal

The new valve was located between the trash rack and the intake pipe flange at Elevation 217.5. Two different valve cases were analyzed – a 78-inch knife gate valve weighing 55 kips with a vertical dimension of 15 inches and a horizontal eccentricity of 2.3 feet, and a 60-inch butterfly valve weighing 35 kips with a vertical dimension of 15 inches and two reducers with vertical dimension of 3 feet each and no horizontal eccentricity.

FINITE ELEMENT MODEL

The general purpose finite element program ANSYS[3] was used for the analysis. The analysis was performed using large displacement, nonlinear material properties for the pipe steel and nonlinear soil springs.

The finite element model geometries for the two different valves under consideration are shown in **Figure 5**.

Model Geometry

The portion of the intake portal above elevation 187.21 feet was modeled and included the intake pipe, the valve, and the trash rack.

The intake pipe was modeled using the SHELL143 element from the ANSYS element library. The element has six degrees of freedom (three translational and three rotational) at each node and has plasticity, creep, stress stiffening, large deflection, and small strain capabilities. The element has linear shape function in both in-plane directions and a mixed interpolation of torsional components in the out of plane direction.

The valve ring was modeled using the SOLID45 element from the ANSYS element library. The element is defined by eight nodes with three translational degrees of freedom at each node.

The trash rack structure was modeled using the BEAM24 element from the ANSYS element library. BEAM24 is a uniaxial element of arbitrary cross-section (open or closed) with tension-compression, bending and St. Venant torsional capabilities. The element has six degrees of freedom (three translational and three rotational) at each node.

The trash rack dome and the reducers for the 60-inch butterfly valve case were modeled using elastic shell element because nonlinearity is not expected for these components. For this reason, the SHELL63 element from the ANSYS element library was used. SHELL63 has six degrees of freedom (three translational and three rotational) at each node and has both bending and membrane capabilities. The element also has stress stiffening and large deflection capabilities.

The 78-inch knife gate valve model has 13,542 degrees of freedom and 2,293 nodes. A total of 2,196 shell elements, 36 solid elements and 60 beam elements were used to define this model. The 60-inch butterfly valve model has 15,270 degrees of freedom and 2,581 nodes. The model was defined by 2,484 shell elements, 36 solid elements and 60 beam elements.

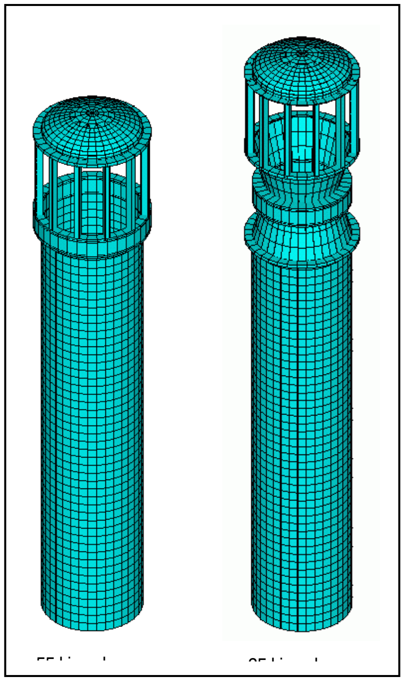


Figure 5: Finite Element Model Geometry (78-inch knife gate [left], 60-inch butterfly [right])

Material Properties

The intake pipe was modeled with a bilinear material model to represent ASTM A570 Grade C steel. Steel properties included 33.4 ksi yield strength, 52.2 ksi ultimate strength, and 23% elongation at failure[4]. Linear elastic material was used for the trash rack dome and reducers.

Soil Springs

The surrounding soil was modeled using nonlinear p-y (horizontal) and t-z (axial) springs. The p-y springs were modeled as compression only with no tension capability. The t-z springs were used to model friction at the pipe-soil interface. The soil spring data was provided by the geotechnical consultant (GeoPentech, Inc.). The p-y and t-z curves from GeoPentech and the idealized multi-linear curves used in the ANSYS model are shown in **Figure 6**.

Within ANSYS, the soil springs were modeled using the COMBIN39 nonlinear spring element from the ANSYS element library. COMBIN39 is a unidirectional element with nonlinear generalized force-deflection capability. The spring elements were distributed at 10 degree intervals along the perimeter of the pipe model. Spring value at each node was computed such that the total horizontal compression component of all springs for any horizontal direction is equal to the horizontal spring value provided by the geotechnical engineer. The vertical springs, specified as force per unit area by the geotechnical engineer, were multiplied by the tributary area for each node to define the spring stiffness at each node in the ANSYS model.

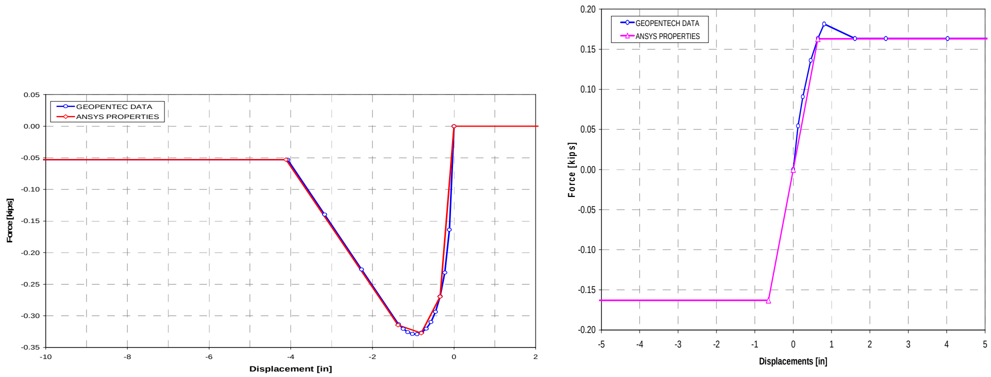


Figure 6: Soil Springs used in the FE Model

Boundary Conditions

All nodes at the pipe-concrete interface (elevation 187.21) were assumed to be fixed against translation and rotation because the pipe is fully encased in a massive reinforced concrete block. The portal was also assumed to be fixed against movement because it is shear keyed into the bedrock.

Seismic Loading

Median plus one standard deviation estimate of ground motions from a Magnitude 7.25 earthquake on the Hayward fault were used for the analysis of the intake pipe[5]. Based on this scenario earthquake, the design horizontal and vertical peak ground acceleration (PGA) being used for the seismic improvement work at the dam embankment are 0.961g and 1.043g, respectively.

Acceleration time histories along two orthogonal and the vertical direction for use in the finite element analysis of the intake pipe were provided by the geotechnical consultant. Acceleration time histories from the Magnitude 7.7, September 16, 1978 Tabas, Iran earthquake were spectrally matched to the design response spectra for the dam embankment and represent motions along the transverse, longitudinal, and vertical axis of the dam embankment.

The design response spectra and the spectra of the matched acceleration time histories are shown in **Figure 7**. The figure shows that for periods less than about 0.75 seconds, the two horizontal spectra are identical, whereas for periods greater than 0.75 seconds, the dam longitudinal spectrum is greater than the dam transverse spectrum. The transverse axis of the dam is oriented 26 degrees relative to the strike of the Hayward fault and the dam longitudinal and the dam transverse do not represent the fault normal and fault parallel motions. However, since the fundamental period of vibration of around 0.2 seconds (Table 1) for the intake pipe was significantly less than 0.75, the difference between the fault normal and fault parallel motions was not considered to significantly impact the seismic response of the intake pipe; therefore, the dam longitudinal and dam transverse motions were used in the analysis together with the dam vertical motions.

The spectrum matched horizontal and vertical acceleration time histories used in the analysis are shown in **Figure 8**.

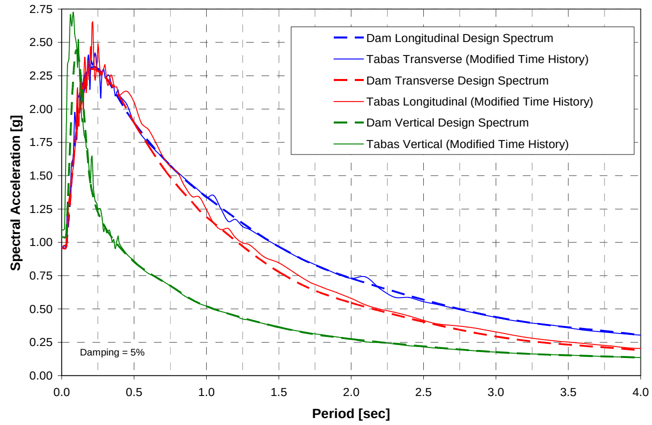


Figure 7: Design Response Spectra

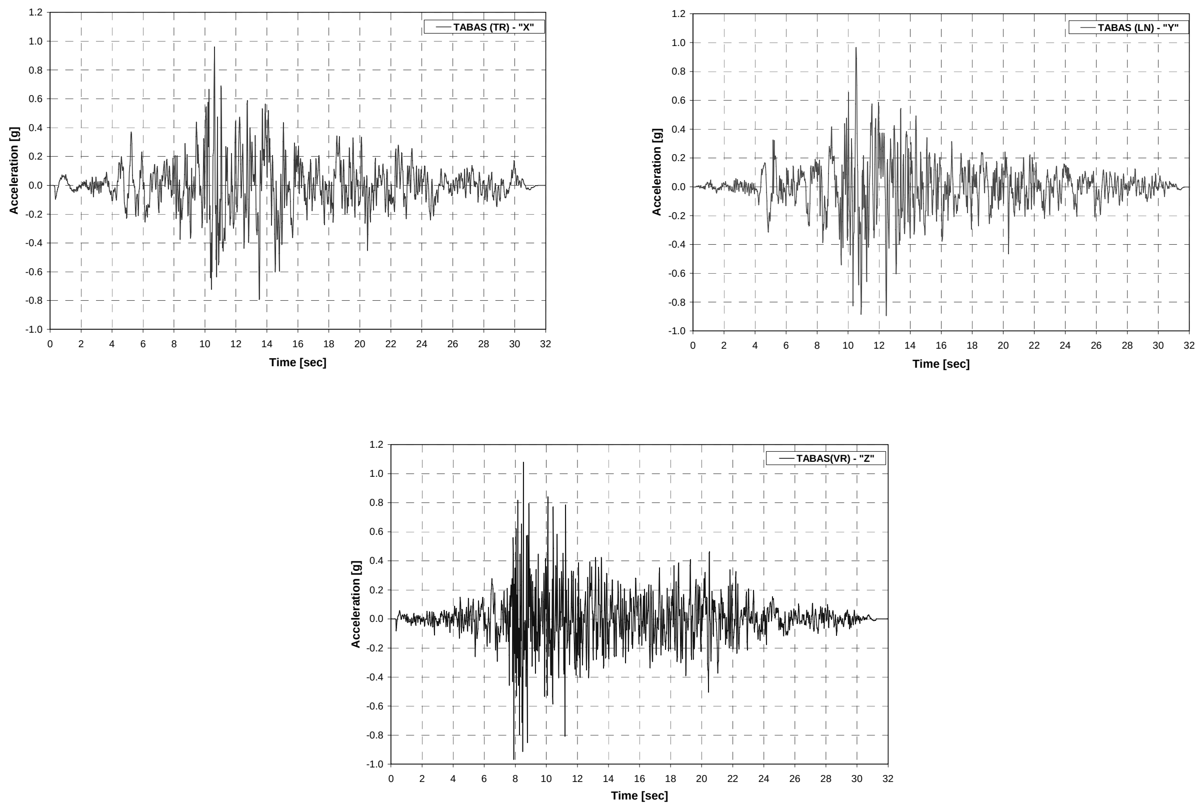


Figure 8: Dam-Transverse, Dam-Longitudinal and Dam-Vertical Acceleration Time Histories

Hydrodynamic Loading

The hydrodynamic effect of water was modeled using added lumped masses to the pipe. The resultant hydrostatic pressure acting on the inner and outer cylindrical surfaces of the pipe is zero; thus, no hydrostatic water loads were applied. However, the hydrodynamic loading due to seismic excitation was included by added-mass terms combined with the mass of the steel intake pipe. The reservoir water is conservatively assumed at the spillway crest elevation of 313.7 feet. The hydrodynamic added mass is calculated in accordance with US Army Corps of Engineers Engineer's Manual EM 1110-2-2400[6,7]. **Figure 9** shows the methodology used for the computation of the hydrodynamic added mass.

The added mass of each vertical member of the trash rack (TS6x4x03125) was calculated assuming a circular section with same area and the resulting mass was doubled to account for grating. The added mass of the dished head of the trash rack was computed by representing it with a series of short cylinders, as shown in **Figure 10**.

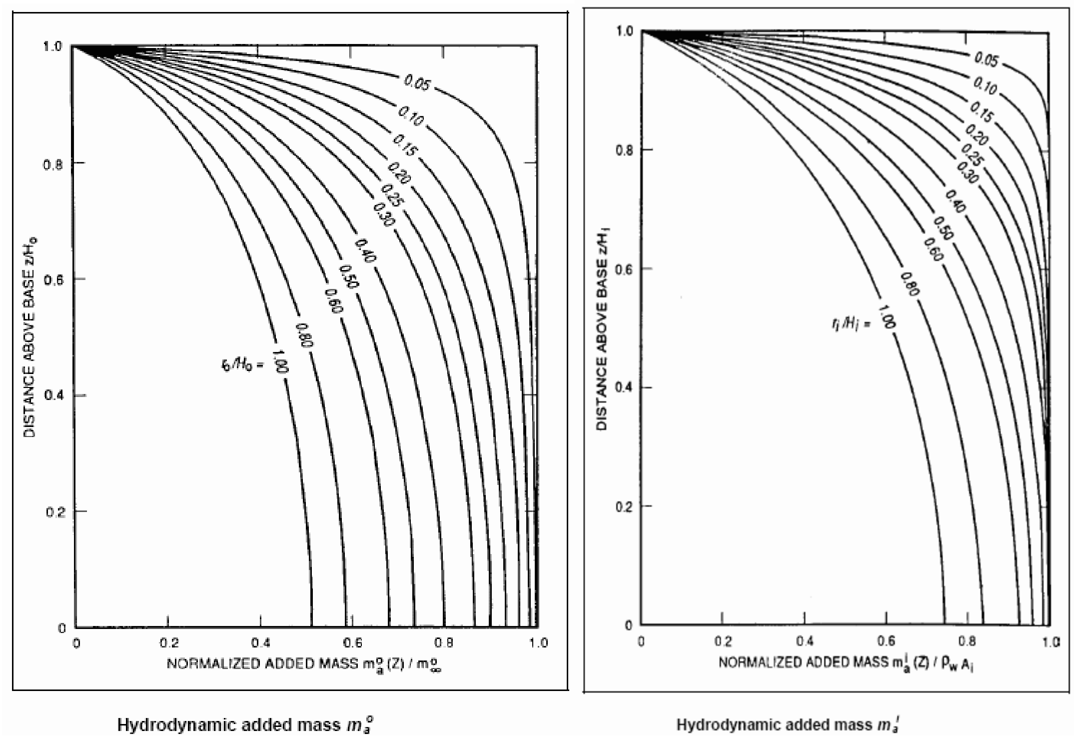


Figure 9: Computation of Hydrodynamic Mass

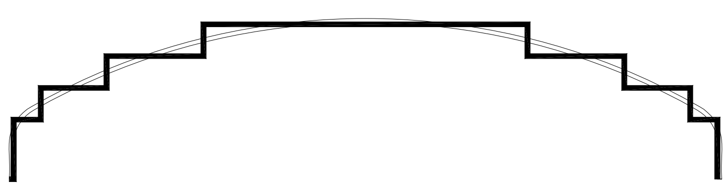


Figure 10: Idealized representation of dished head for the computation of hydrodynamic loads

ANALYSIS RESULTS

Nonlinear finite element analysis of the intake pipe for two different valve configurations was performed. Table 1 shows the periods of vibration for the most significant modes of vibration for the two valve configurations.

Table 1: Vibration Period for Different Modes

Analysis Case	Mode No.	Frequency [Hz]	Period [sec]	Mass Fraction [%]		
				"X" Direction	"Y" Direction	"Z" Direction
60-inch Butterfly (35 Kip)	1	4.16	0.240	2.4%	63.6%	0.0%
	2	4.16	0.240	63.6%	2.4%	0.0%
	3	10.012	0.100	0.2%	3.4%	0.0%
	4	10.012	0.100	3.4%	0.2%	0.0%
	15	25.23	0.040	0.1%	19.7%	0.0%
	16	25.23	0.040	19.7%	0.1%	0.0%
	39	52.772	0.019	0.0%	0.0%	99.8%
	46	58.526	0.017	0.0%	6.1%	0.0%
	47	58.526	0.017	6.1%	0.0%	0.0%
78-inch Knife Gate (55 Kip)	1	4.597	0.218	0.0%	70.0%	0.0%
	2	4.611	0.217	70.0%	0.0%	0.4%
	7	15.96	0.063	0.0%	2.5%	0.0%
	8	16.064	0.062	3.1%	0.0%	0.1%
	17	27.268	0.037	6.9%	0.0%	2.4%
	18	27.336	0.037	0.0%	6.3%	0.0%
	19	29.294	0.034	9.7%	0.0%	6.8%
	20	30.338	0.033	0.0%	8.4%	0.0%
	29	40.698	0.025	0.2%	0.0%	9.6%
	34	43.509	0.023	0.0%	1.2%	0.0%
	35	44.735	0.022	1.8%	0.0%	76.9%

The fundamental periods of vibration for the 60-inch butterfly and the 78-inch knife gate valve cases are 0.24 seconds and 0.22 seconds, respectively, in both the X and Y directions. The most significant vertical mode of vibration has a period of 0.019 seconds for the 60-inch valve case and 0.022 seconds for the 78-inch valve case. Periods for the other significant modes of vibration for the two cases are included in Table 1. The deformed shape for the fundamental modes of vibration for the two cases is shown in Figure 11.

The results of the analysis show that for both valve configurations the maximum principal stresses were below the 33.4 ksi yield strength for the ASTM A570 Grade C material. The maximum principal stresses for the 60-inch butterfly and the 78-inch knife gate valve cases were 27.16 ksi and 26.99 ksi, respectively. The results showed slightly higher stresses for the 60-inch butterfly valve, even though it is 20 kips lighter than the 78-inch knife gate and the spectral acceleration associated with its fundamental period of vibration is less than that for the 78-inch knife gate (**Figure 7**). This is due to several factors that include the additional weight of water within the two 3-feet tall reducers and the higher center of mass (because of the reducers) for the 60-inch butterfly valve. These effects compensate for the higher weight, horizontal eccentricity, and higher spectral acceleration of the 78-inch knife gate.

The analysis also showed no global buckling or wrinkling of the pipe. **Figure 12** shows the maximum stress plots of the pipe for the two valve configurations. Although the bolt material for the existing bolts at the pipe-valve connection was not known, the total tension and shear forces for each bolt were well below the conservatively assumed bolt capacities.

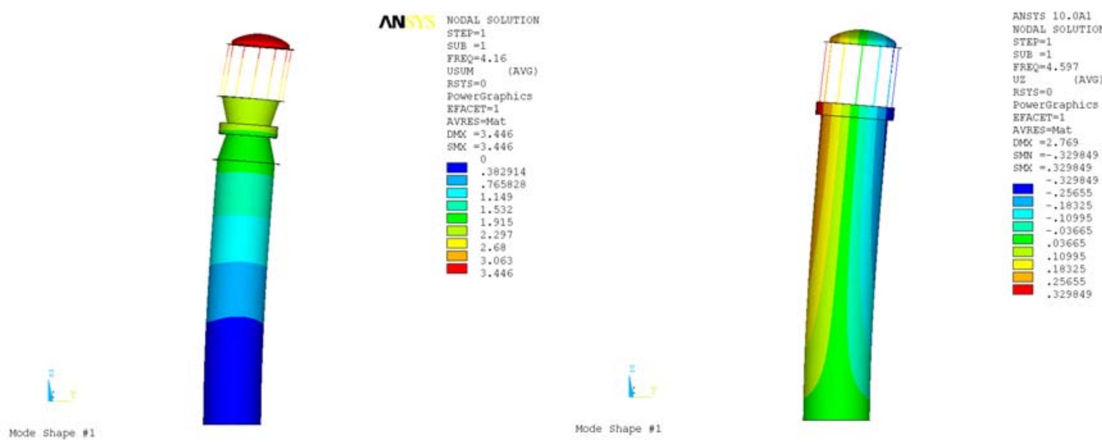


Figure 11: Fundamental Mode Shape (35Kip Case [left], 55Kip Case [right])

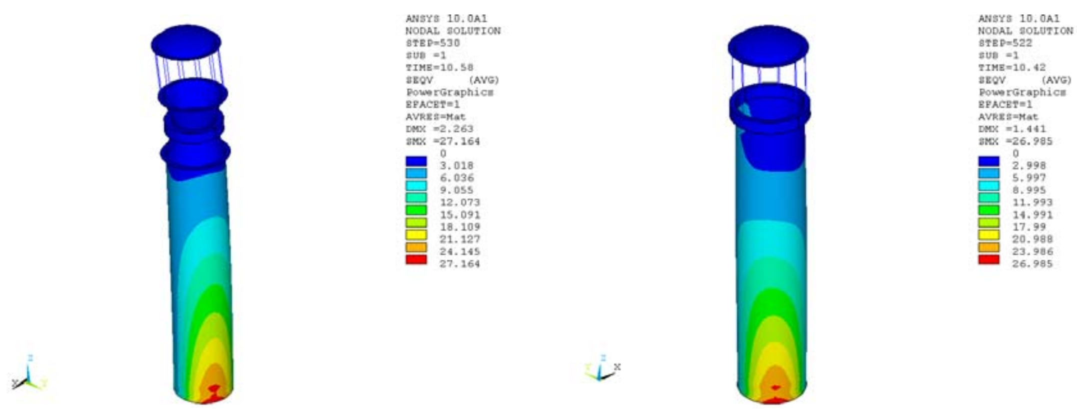


Figure 12: Maximum Principal Stress (35Kip Case [left], 55Kip Case [right])

CONCLUSIONS

This paper presents finite element analysis that was performed to help the District confirm the location of and decide between different valve configurations for the low level outlet. The decision had to be made in a three week time frame because of the requirement to pre-purchase the valve, since it was a significant capital expense (approximately three quarters of a million dollars) requiring Board approval, and meet a nine week construction window. Any delay would have impacted the larger $60 million embankment upgrade project. A functioning low level outlet was essential prior to embarking on the embankment upgrade project in case the reservoir needed to be drained in an emergency.

A sophisticated nonlinear analysis was performed for the low level outlet intake pipe, a 30-foot tall, 78inch diameter fully submerged cantilever, located approximately two kilometers from a major active fault capable of generating a Magnitude 7+ earthquake. The analysis assessed the impact of adding up to 55,000 lbs of heavy valve on the intake pipe, which was not originally designed to carry this extra load.

The results of the analysis showed that the maximum stress induced in the pipe was approximately 80% of the nominal yield strength of the pipe. The analysis also showed that the addition of the new valve did not result in any local or global buckling of the pipe under static or seismic loading, thereby providing the District the necessary confidence to proceed with the project.

Based on the results of the analysis, the District decided to use a smaller 60 inch knife gate with reducing flanges. The weight of the valve was less than that for the 78 inch knife gate analyzed in this study.

ACKNOWLEDGMENTS

Work presented in this paper was part of the embankment upgrade project. Engineering staff from GeoPentech, Inc. including Mr. John Barneich, Dr. Phalkun Tan and Dr. Yoshi Moriwaki provided the necessary geotechnical input. Ms. Guilaine Roussel of Terra Engineers provided logistical support as the overall project manager of the design consulting team. Mr. Atta Yiadom, the District's project manager , and Mr. Sean Todaro, District's project engineer, provided the necessary input and support to complete the project within the ambitious three week time period.

REFERENCES

[1]. [1] "Replacement of the Low-Level Outlet Control at San Pablo Dam", by Sean M. Todaro and Ray G. Wong, United States Society on Dams 29th Annual Meeting and Conference, April 2006.

[2]. [2] The Uniform California Earthquake Rupture Forecast, Version 2 (UCERF 2), by 2007 Working Group on California Earthquake Probabilities, USGS Open File Report 2007-1437, CGS Special Report 203, SCEC Contribution #1138, Version 1.0, 2008

[3]. [3] ANSYS Release 10.0, ANSYS Inc., Southpointe, 275 Technology Drive, Canonsburg, PA 15317, www.ansys.com

[4]. [4] American Institute of Steel Construction, "Manual of Steel Construction-Allowable Stress Design", 9th Edition, 1989.

[5]. [5] "Dam Draft Design Memorandum", Design of San Pablo Dam Seismic Upgrades: DM-01 Seismic Design Criteria-Revised dated 1/29/07.

[6]. [6] US Army Corps of Engineers, "Time-History Dynamic Analysis of Concrete Hydraulic Structures", EM 1110-2-6051, 22 December 2003.

[7]. [7] US Army Corps of Engineers, "Structural Design and Evaluation of Outlet Works", EM 1110-2-2400, 2 June 2003.

Participants List

Japan Participants

Munetaka Abe
Assistant Chief of Engineering Division
Japan Water Works Association
m.abe@jwwa.or.jp

Mikita Amano
Japan Water Research Center
amano@jwrc-net.or.jp

Kazuhiko Fujimura
Tokyo Waterworks Service Co., Ltd.
fujimura-kazu@tssk.jp

Nobuhiro Hasegawa
Pipeline Engineer, Water Pipeline Department
JFE Engineering Corp
hasegawa-nobuhiro@jfe-eng.co.jp

Naoki Hosoi
Tokyo Waterworks Service Co., Ltd.
hosoi-naoki@tssk.jp

Yoshinori Itani
Kubota Corporation
yo-itani@kubota.co.jp

Masao Kadowaki
Hanshin Water Supply Authority
kadowaki-m@hansui.or.jp

Toshichika Matsui
Waterworks & Sewerage Bureau, City of Nagoya
haisui@jogesuido.city.nagoya.lg.jp

Masakatsu Miyajima
Professor
Kanazawa University
miyajima@t.kanazawa-u.ac.jp

Noboru Murakami
Group Leader, Management Planning Division, Synthesis Policy Group
Hachinohe Water Supply Authority
hassui02@jomon.ne.jp

Hiroshi Nagaoka
Professor, Department of Urban and Civil Engineering
Tokyo City University
hnagaoka@tcu.ac.jp

Kazutomo Nakamura
Senior Engineer, Training and International Department
Japan Water Works Association
k.nakamura@jwwa.or.jp

Hayato Nakazono
Tokyo Engineering Consultants
nakazono-h@tokyoengicon.co.jp

Tatsuo Ohmachi
Professor, Interdisciplinary Graduate School of Science and Engineering
Tokyo Institute of Technology
ohmachi@enveng.titech.ac.jp

Kimiyasu Ohtake
Nippon Jogesuido Sekkei, Co., Ltd.
kimiyasu_ohtake@njs.co.jp

Yuki Sagara
Osaka Municipal Waterworks Bureau
yu-sagara@suido.city.osaka.jp

Akira Suzuki
Bureau of Waterworks, Tokyo Metropolitan Government
suzuki-akira01@waterworks.metro.tokyo.jp

Yasusi Taguchi
Japan Water Works Association
y.taguchi@jwwa.or.jp

Toru Tomioka
Japan Water Works Association
ttomioka@jwwa.or.jp

Hayato Yokono
Kobe Municipal Waterworks Bureau
hayato_yokono@office.city.kobe.lg.jp

Taiwan Participants

K. P. Chang
Sub-section Chief
Taipei Water Department
castor@twd.gov.tw

Jerry Jwo-Ran Chen
Senior Geotechnical Engineer, Dept. of Geotechnical Engineering
CECI Engineering Consultants, Inc., Taiwan
jjchen@ceci.com.tw

Jiin-Shyang Chen
Chief Engineer
Taipei Water Department
tw110529@twd.gov.tw

Shen-Hsien Chen
Director
Water Resources Agency, Ministry of Economic Affairs, Taiwan
lv@wra.gov.tw

Chin-Tse Cheng
Senior Engineer& Manager
Taipei Water Department
ctcheng@twd.gov.tw

Y. Chuan Chou
Deputy Manager, Dept. of Geotechnical Engineering
CECI Engineering Consultants, Inc., Taiwan
cwt39845@ceci.com.tw

Yu-Chih Chou
Assistant
Chinese Taiwan Water Works Association
ctwwa@ctwwa.org.tw

Pei-Chung Hsu
Secretary General
Chinese Taiwan Water Works Association
hsu1031@ctwwa.org.tw

Hsiang-Yuan Hung
Assistant Researcher
National Center for Research on Earthquake Engineering, Taiwan
hyhung@ncree.org.tw

Nan-Tzer Hwu
Vice-President
Taiwan Water Corporation
hunantzer@mail.water.gov.tw

Chaur-Gong Jong
Chief, Water Resources Management Division
Water Resources Agency, Ministry of Economic Affairs, Taiwan
a620010@ms1.wra.gov.tw

Hann-Hsiang Ju
Sub-section Chief, Engineering Division
Taipei Water Department
hann@twd.gov.tw

Siao-Syun Ke
Assistant Researcher
National Science & Technology Center for Disaster Reduction, Taiwan
opbook@ncdr.nat.gov.tw

Tsung-Shen Liao
President
Chinese Taiwan Water Works Association
president@ctwwa.org.tw

Chen-Che Lin
Associate Engineer, Water Resources Management Division
Water Resources Agency, Ministry of Economic Affairs, Taiwan
a620250@ms1.wra.gov.tw

S. C. Lin
Engineer
Taipei Water Department
archy@twd.gov.tw

Karl Gee-Yu Liu
Associate Research Fellow
National Center for Research on Earthquake Engineering, Taiwan
karl@ncree.org.tw

Ban-Jwu Shih
Associate Professor, Department of Civil Engineering
National Taipei University of Technology
bjshih@ntut.edu.tw

Shu-Yuh Shih
Director, Department of Water Supply
Taiwan Water Corporation
shih486@mail.water.gov.tw

S. J. Shue
Engineer
Taipei Water Department
shue@twd.gov.tw

S. L. Teng
Engineer
Taipei Water Department
tsl@twd.gov.tw

Jiin-Song Tsai
Professor, Department of Civil Engineering
National Cheng Kung University
jstsai@mail.ncku.edu.tw

Bing-Xin Wang
Manager, Eighth Branch
Taiwan Water Corporation
1200@mail.water.gov.tw

Yung-Ming Wang
Engineer
Taipei Water Department
steve@twd.gov.tw

Yang-Long Wu
Superintendent, Engineering Division
Taipei Water Department
good@twd.gov.tw

US Participants

Luke Cheng
Manager, Structural Design Section, Engineering Management Bureau
San Francisco Public Utilities Commission
lcheng@sfwater.org

Shonnie Cline
Senior Account Manager
Water Research Foundation
scline@WaterResearchFoundation.org

Craig Davis
Waterworks Engineer, Geotechnical Engineering Group
Los Angeles Department of Water and Power
craig.davis@ladwp.com

John Eidinger
President
G&E Engineering Systems Inc.
eidinger@geEngineeringSystems.com

Xavier Irias
Director of Engineering and Construction
East Bay Municipal Utility District
xirias@ebmud.com

David Lee
Senior Civil Engineer
East Bay Municipal Utility District
dlee@ebmud.com

Wally Lieu
Engineering Section Manager
Metropolitan Water District of Southern California
wlieu@mwdh2o.com

Mario Rosario Mazzola
Professor, School of Engineering,
Università di Palermo, Palermo, Italy
mazzola@idra.unipa.it

David Pratt
Manager of Design
East Bay Municipal Utility District
dpratt@ebmud.com

Masanobu Shinozuka
Professor and Chair, Department of Civil and Environmental Engineering
University of California at Irvine
shino@uci.edu

Keith Walker
Senior Engineer
Portland Water Bureau
Keith.Walker@ci.portland.or.us

Steve Welch
Director of Engineering
Contra Costa Water District
swelch@ccwater.com

Chandrika Winston
Manager, Gas and Water Engineering Department
Memphis Light, Gas and Water
cwinston@mlgw.org