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Reliability and Restoration of Water Supply Systems for Fire Suppression and Drinking Following Earthquakes

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NIST

United States Department of Commerce
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U.S. Department of Commerce
Technology Adminstration
National Institute of Standards and Technology
Building and Fire Research Laboratory
Gaithersburg, MD 20899

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U.S. Department of Commerce
William M. Daley, *Secretary*
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PREFACE

The Congressional emergency appropriation resulting from the January 17, 1994 Northridge earthquake provided the Building and Fire Research Laboratory (BFRL) at the National Institute of Standards and Technology (NIST) an opportunity to increase its activities in earthquake engineering under the National Earthquake Hazard Reduction Program (NEHRP). In addition to the post-Northridge earthquake reconnaissance, BFRL concentrated its efforts primarily in the study of post-earthquake fire and lifelines, and moment resisting steel frames.

BFRL sponsored a post-earthquake fire and lifelines workshop in Long Beach, California in January 1995 to assess technology development and research needs that will be used in developing recommendations to reduce the effects of post-earthquake fires. The workshop participants developed a list of priority project areas where further research, technology development, or information collection and dissemination would serve as a vital step in reducing the losses from post-earthquake fires. NIST funded a number of studies identified by the participants which are listed in NIST Special Publication 889.

BFRL, working with practicing engineers, carried out surveys and assessment of the damaged buildings and partially funded a SAC (Structural Engineers Association of California, Applied Technology Council, California Universities for Research in Earthquake engineering) workshop on seismic performance of steel frame buildings in September 1994. The objectives of the workshop were threefold: 1) to coordinate related interests; 2) focus on the problems observed in the performance of steel buildings; and 3) develop a research plan to solve the problems. NIST funded the research and engineering communities to carry out several of the proposed studies.

This report represents a part of these studies related to post-earthquake fire and lifelines sponsored by NIST as part of the Congressional emergency appropriation.

ABSTRACT

In the Kobe, Northridge, and Loma Prieta earthquakes, the water systems providing water for fire suppression failed. In the case of Kobe, a significant investment had been made to mitigate earthquake damage. This report provides an overview of post-earthquake system reliability, and makes recommendations to enhance post-earthquake operability of domestic water systems and/or alternate water supply systems and/or enable quick restoration of service following an earthquake.

This report and associated projects are the result of recommendations from the Post-Earthquake Fire and Lifeline Workshop held in Long Beach, California, January 30 - 31, 1995. The report includes information and procedures, developed from previous research and observations, to assess earthquake reliability of municipal water supplies focusing on water for fire suppression. The relevant information base includes experience from previous earthquakes, overall system operation, and performance of individual system components. Component and system failure mechanisms are also addressed, and mitigation approaches are identified and developed.

The report also considers alternatives to enhance restoration of service following earthquakes. Alternatives include hardware improvements, system operation optimization, development of alternative water supplies, and use of geographic information systems, GIS.

UNITS

Because much of the information presented in this report is taken from original sources, no attempt has been made to convert units of measurement such that all units are SI or English. Both units of measurement appear throughout the report. Rather than adopt one set of units as the standard, a list of conversion factors are given below for the units used in the report.

SI		English	English		SI
1 cm	=	0.394 in	1 in	=	2.54 cm
1 m	=	3.28 ft	1 ft	=	0.305 m
1 km	=	0.622 mi	1 mi	=	1.61 km
1 gal = 1 cm/s^2	=	0.394 in/s^2	1 in/s^2	=	2.54 cm/s^2 = 2.54 gal
1 liter	=	0.264 gallons	1 gallon	=	3.79 liters
1 liter/min	=	0.264 gpm	1 gpm	=	3.79 liters/min
1 mg/l	=	8.35 x 10^{-6} lb/gallon	1 lb/gallon	=	0.120 kg/l
1 N	=	0.225 lb	1 lb	=	4.45 N
1 kPa	=	0.145 psi	1 psi	=	6.89 kPa
1 hectare	=	2.47 acres	1 acre	=	4.047 m^2 = 0.4047 hect.

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The report was prepared by the authors who served as the Principal investigators for the study. The primary authors of the chapter are:

1. Introduction
   D. Ballantyne and F. Blackbum
2. Earthquake Hazards
   C. B. Crouse
3. Performance of Water Systems in Historic Earthquakes
   D. Ballantyne
4. Performance Criteria
   D. Ballantyne
5. Water System Reliability
   W. Heubach and N. Basoz
6. System Component Damage
   M. O'Rourke, P. Summers, and A. Nisar
7. Mitigation Alternatives
   D. Ballantyne, F. Blackbum, and P. Summers
8. Conclusions and Recommendations
   D. Ballantyne

Appendix A is a collection of papers documenting the performance of water systems during 8 earthquakes, 1 firestorm and 1 flood. Several authors of the Chapters of the body of the report, plus J. Eidinger, C. Scawthom, and T.D. O'Rourke, prepared these Appendix A papers.

TABLE OF CONTENTS

APPENDIX A

TABLES

FIGURES

EXECUTIVE SUMMARY

OVERVIEW

Following the 1989 Loma Prieta and 1994 Northridge earthquakes in California, and the 1995 Kobe, Japan earthquake, major system water supply was lost for fire suppression and for domestic use. None of these recent events turned out to be as catastrophic as the fire storms following the 1906 San Francisco, California, and 1923 Kanto, Japan, earthquakes. The classic scenario occurred in the Loma Prieta, Northridge, and Kobe earthquakes where pipelines in both the transmission and distribution system failed. Reservoirs drained, and there was no water available for fire suppression or domestic use.

This report helps quantify the problem of water system reliability. It identifies system and component failure in historic events, and summarizes the results. Vulnerability evaluation techniques are developed for pipelines and reservoirs. Mitigation measures for system and components are discussed along with other alternatives such as developing alternative water supplies, optimizing system control, and use of geographic information systems (GIS).

This report, and the associated project is the result of recommendations from the Post-Earthquake Fire and Lifeline Workshop held in Long Beach, California, January 30 - 31, 1995. It includes research results to assess earthquake reliability of municipal water supplies focusing on water for fire suppression. It considers experience from previous earthquakes, overall system operation, and performance of individual system components. Component and system failure mechanisms are addressed, and mitigation approaches identified and developed.

MUTUAL NEEDS OF WATER AND FIRE DEPARTMENTS

Fire departments and water purveyors are in many cases not part of the same governmental jurisdiction. However, fire departments are highly dependent on adequate water supplies to function effectively. Large fires reaching conflagration proportions have become common place. In many instances, water supply to meet large fire flow demands is insufficient, and liaison between water agencies and fire departments is poor or ineffective.

Legal and financial authorization governing the raising of capital to pay for facilities can be inconsistent with fire department needs. It is common for state regulations to require some minimum levels of fire flows for new facilities. As fire flow demands change, it may be difficult for water agencies to legally justify charging rate payers to build facilities to enhance fire flows.

Coordination between water and fire agencies must take place during pre-event planning, and should include operational procedures and priorities. Fire departments should attain a good working knowledge of the water system they use, and understand the earthquake vulnerability of that system.

EARTHQUAKES HAZARDS

Fault rupture results in earthquakes with a measured magnitude, and site-specific intensity. Fault offsets can also cause tsunamis. Ground motion, one measure of earthquake intensity, is often measured in terms of peak ground acceleration. Earthquake shaking can result in soil liquefaction and landslides that can be particularly damaging to buried water system components.

PERFORMANCE OF WATER SYSTEMS IN HISTORIC EARTHQUAKES

Historic performance of water systems in earthquakes, as well as two instances of non-earthquake-related water system failure, were reviewed to identify common earthquake deficiencies that repeatedly occur, resulting in dysfunction of water systems. Hazard events reviewed include:

1. Key earthquakes - Kobe, Japan, 1995, Northridge, California, 1994; and, Loma Prieta, California, 1989.

2. Less destructive earthquakes that caused water system damage - Landers/Big Bear, California, 1992; Cape Mendocino, (Petrolia), California, 1992; and, Whittier, California, 1987.

3. Historic devastating earthquakes - Kanto, Japan, 1923; and, San Francisco, California,1906.

4. Potable water system outage from other natural disasters - Des Moines, Iowa, Flood, 1993, 12 days water outage resulting from flooding of a water treatment plant, and Oakland, California, Hills Fire, 1991, exacerbated by inadequate water supply.

There is a correlation between incidents where there was inadequate water for fire suppression, and where fire became a significant issue. The only two events where fire was a small or non-issue were Whittier, which was a relatively small earthquake, and Landers, where there is a sparse population and building density. There is also a strong correlation between the three most significant fires, San Francisco in 1906, Kanto, and Kobe, and the ineffective use/unavailability of an alternate water supply. System component failures can be grouped by significance of impact on system dysfunction as follows:

- Very High Impact - Pipe damage due to permanent ground deformation (PGD).

- High Impact – Pipe damage due to wave propagation and raw water transmission pipeline failure.

- Moderate Impact - water treatment plant damage, loss of power, and tank inlet/outlet pipe damage.

- Low Impact - tank shell/structure damage, surface supply failure, and well casing and equipment damage.

We conclude that pipeline damage due to PGD and wave propagation, in both transmission and distribution systems, had the greatest impact in most of these events.

PERFORMANCE CRITERIA

Performance criteria are needed to define the desired level of post-earthquake service. It ties five categories of water system performance to the probability of the defined earthquake ground motion being exceeded. Earthquake ground motions with a 50 percent chance of being exceeded (72 year return period) and a 10 percent chance of being exceeded (475 year return period) in 50 years are set as reference points. Performance criteria for these two probabilities are shown in Table ES-1.

TABLE ES-1
WATER PERFORMANCE OBJECTIVES -
ACCEPTABLE ADVERSE CONSEQUENCE LEVELS FOR TWO EARTHQUAKE LEVELS

PERFORMANCE CATEGORY	ACCEPTABLE ADVERSE CONSEQUENCES OBE (50% chance in 50 years)	DBE (10% chance in 50 years)
Life Safety	Minimal – Injury or loss of life are not acceptable consequences	Minimal - Injury or loss of life are not acceptable consequences
Fire Suppression	Minimal - With the exception of small isolated areas that are not densely populated, water for fire suppression should be available for entire service area.	Moderate - Water for fire suppression should be available for a minimum of 70% of the service area including all industrial areas and densely populated business and residential areas.
Public Health	Low - Water should be available for all but a few isolated areas. Boil water order acceptable for up to 48 hours.	Moderate - Provide service for at least 50% of system. Boil water order, or delivery by tanker truck acceptable. Restore100% service in 1 week.
System Restoration	Low - Water should be available for all but a few isolated areas.	Moderate - Service should be available for at least 50% of system. Restoration to 100% service within one week.
Property Damage	Low - Any damage should not affect facility functionality and should be repairable.	Moderate - 100% loss of nonessential facilities acceptable if not cost-effective to upgrade, and other performance objectives are met.

RELIABILITY ANALYSIS

There are a variety of approaches that can be used to assess post-earthquake water system reliability:

- Conduct deterministic assessments of each water system component and use the component assessment results to develop a system performance scenario. Typically, three steps are involved. First, the seismic hazards are defined. Based on the seismic hazards and component characteristics, component vulnerability is then determined. The final step is use the component vulnerabilities to predict overall system performance.

- Express component vulnerability in probabilistic terms and use probabilistic techniques to evaluate system reliability. There is a significant amount of uncertainty associated with earthquake hazards and the response of facilities subjected to earthquake hazards. Although accurately modeling this uncertainty is difficult, probabilistic assessments can be used to assess the magnitude and likelihood of variations from the expected outcome.

- Use system reliability assessment techniques such as fault tree analysis. Although fault trees have been used extensively in many applications such as the nuclear industry, fault trees have not been used as extensively by water utilities. Fault trees can be used to calculate failure probabilities, identify paths that may lead to failure, and to identify those events that are most likely to lead to failure.

- Use GIS, which allows calculation and graphic presentation of system risk assessment results that can be easily used and interpreted by planners, emergency response personnel and engineers. Pipeline construction materials and joint types can be electronically overlaid on the earthquake hazards. Damage algorithms, that relate pipe damage to ground shaking intensity and permanent ground displacement, can be used to determine pipe vulnerability using the GIS. Pipe criticality can be determined, and considered in the risk assessment determination.

SYSTEM COMPONENT DAMAGE

Damage to the key water system components including pipelines, reservoirs, treatment plants, and pump stations has resulted in water system failures.

Pipelines

The Northridge earthquake caused 1,500 pipeline failures in the Los Angeles system. The Kobe earthquake caused 1,600 distribution system failures. In both earthquakes, systems were quickly drained through damaged pipe, rendering the systems dysfunctional in many locations. Pipe joint damage is predominant for large diameter segmented pipe. For smaller diameter pipe, pipe barrels fail in addition to joint damage. Welded steel pipe damage sometimes fails due to localized wrinkling from compression, often concentrated at welded joints.

Analytical techniques have been developed to assess both continuous (welded joint) and segmented (bell and spigot joint) pipelines. Permanent ground deformation (PGD), often due to liquefaction/lateral spread, is the usual cause of damage to continuous welded steel pipelines. The steel's ductility helps minimize pipe damage due to wave propagation. Brittle segmented pipe with rigid joints, such as cast iron with leaded joints, is one of the most vulnerable segmented pipe systems, and is vulnerable to both wave propagation and PGD. Use of ductile materials such as ductile iron, and joint restraint, greatly enhances pipe earthquake performance. Relationships between earthquake intensity parameters such as peak ground velocity and PGD, and unit failure rates have been developed for a variety of pipe materials, based primarily on empirical data. These relationships can be used to estimate pipeline damage for given earthquake scenarios. Damage estimates can be further used to predict water system hydraulic performance.

Reservoirs

Flat bottomed vertical steel water storage reservoirs (tanks) have been damaged in numerous earthquakes. The most common failure mechanism is damaged connection piping. This damage occurs when unanchored tanks rock, and inlet/outlet piping, rigidly attached to the tank and buried in the ground, breaks. In more severe cases, the tanks themselves are damaged. The tank wall lifts off the ground when the tank rocks. The tank wall then impacts the ground when the tank rocks in the opposite direction, causing severe compression loading, and results in wall buckling. This phenomena is often referred to as elephant's foot buckling. Steel tank damage mitigation methods include addition of connecting pipe flexibility, and anchoring the tank to its foundation. On occasion, the tank structure must be strengthened to accommodate seismic loading.

Wire-wrapped concrete tanks are less vulnerable to earthquakes than steel tanks, but have failed. Tanks that were constructed prior to use of "earthquake cables" used in current designs, are the most vulnerable. Corrosion of the wire wrapping is also a concern.

The American Water Works Association tank design standards incorporate seismic design methods. Other analytical methods for analysis of existing tanks have been developed based on historic earthquake performance data.

Treatment Plants and Pump Stations

Water treatment plants and pump stations have common earthquake failure mechanisms including damage to: foundations, process tanks, equipment and piping, electrical power systems, and building structures. Tank structures and large conduits are subject to differential settlement, increased lateral soil pressures, and flotation. Reinforced concrete process tanks have performed well in earthquakes, but immersed elements such as tank baffles are commonly damaged due to earthquake induced hydraulic loading.

Buried yard piping is vulnerable to differential settlement. Inadequate braced plant piping can break due to differential movement, and depending on its contents and location, flood pipe galleries. Pipe flexibility can enhance earthquake performance. Anchored equipment, including office and lab equipment, seldom fail. Loss of power is the most common failure mechanism. Emergency generators are recommended if continuous operation is crucial.

MITIGATION ALTERNATIVES

Effective mitigation strategies to maintain or quickly restore water service following an earthquake include:

- Improve communication between fire and water departments. The objective is to overcome jurisdictional and institutional barriers to optimize water/fire department emergency operations.

- Improve system hardware so it does not fail, such as replacing pipe, upgrading reservoirs, and providing emergency power. Improvement programs can be prioritized considering the importance of each component in operation of the overall system, and the desired performance objectives. Clearly, the vulnerability of each particular component must be considered in the analysis.

- Provide post-earthquake system monitoring and control to isolate damaged sections of the system, so as to allow operation of other undamaged sections. Distribution system piping is vulnerable to earthquakes, and expensive to replace with piping resistant to seismic activity. Systems to monitor post-earthquake system performance, and isolate damaged sections can optimize overall post-earthquake system function.

- Develop alternative sources/supplies such as those dedicated to fire protection. Provision of operational flexibility may provide the best opportunity of being able to provide post-earthquake service. Alternative water supplies to provide water for fire suppression can include: dedicated fire protection systems, portable water supply systems, cisterns distributed throughout the service area, planned use of swimming pools and/or other recreational water-containing facilities, and the planned and tested ability to draft water from local water bodies.

- Further develop application of GIS, as a tool to identify areas of water systems vulnerable to earthquakes, for use by fire suppression response personnel and engineers responsible for system earthquake mitigation. GIS can be used to map earthquake hazards and pipeline inventory. Vulnerability and criticality assessments can be performed within the GIS, and results effectively presented graphically to decision makers.

CONCLUSIONS AND RECOMMENDATIONS

From this study we can conclude the following:

- Immediate post-earthquake water system function is crucial, particularly to suppress fires.

- Water system vulnerability must be understood by both water and fire departments so they can effectively plan for post earthquake response and restoration. Communication between fire and water departments becomes critical.

- Transmission and distribution pipeline damage has been the key element causing water systems to fail in historic earthquakes.

- An important initial step in developing a mitigation program is to define post earthquake performance criteria.

- Water system reliability can be assessed using deterministic, probabilistic, fault tree, and/or GIS techniques.

- Detailed evaluation methods are available for pipelines, reservoirs, and other system components.

Recommended mitigation strategies include:

- Improve the fire and water departments' understanding of the earthquake vulnerability of water systems, and enhance communication between the two organizations.

- Upgrade system hardware, prioritized considering the criticality and vulnerability of each component.

- Provide post-earthquake system monitoring and control to allow rapid isolation of damaged sections of the system.

- Develop alternative sources/supplies of water for fire suppression.

- Use GIS. It is a powerful tool, effective in performing earthquake mitigation programs, and responding to and recovering from earthquake events.

1.0 INTRODUCTION

1.1 OVERVIEW

This report assesses post-earthquake system reliability, and makes recommendations to enhance post-earthquake operability of domestic water systems and/or alternate water supply systems to enable quick restoration of service following an earthquake.

Following the 1989 Loma Prieta and 1994 Northridge earthquakes in California, and the 1995 Kobe, Japan earthquake, major system water supply was lost for fire suppression and for domestic use. None of these recent events were as catastrophic as the fire storms following the 1906 San Francisco, California and 1923 Kanto, Japan earthquakes. Water system failure also occurred following less destructive earthquakes such as the 1992 Landers/Big Bear, 1992 Petrolia, and 1986 Whittier earthquakes in California. Other disasters including the 1991 Oakland Hills, California fire and the 1993 Des Moines, Iowa flood were exacerbated by fire size exceeding the design fire (Oakland Hills) and water system failure (Des Moines).

The classic scenario occurred in the Loma Prieta, Northridge, and Kobe earthquakes. Pipelines in both the transmission and distribution systems failed. Reservoirs drained, and there was no water available for fire suppression or domestic use. As expected, pipeline failures tended to occur where there was permanent ground deformation due to liquefaction (such as in Kobe and Loma Prieta) or tectonic movement (such as in Northridge). Brittle pipelines constructed of cast iron performed worse than ductile pipelines constructed of steel or ductile iron. GIS hazard mapping to identify areas where there will be permanent ground deformation coupled with maps showing pipe materials has become almost a standard practice in the industry to identify vulnerable pipelines.

The Kobe Water Department had an aggressive earthquake mitigation program prior to the earthquake. They had replaced a significant amount of brittle pipe in their system, with 89 percent of the pipe being ductile iron or steel at the time of the earthquake. Nonetheless, there were over 2,000 pipeline failures in the transmission and distribution systems in Kobe and the two adjoining cities that took months to repair. With this background some of the questions addressed in this report are:

- Is water system mitigation effective?
- Can we increase the reliability of post-earthquake water system operation to an acceptable level for an acceptable cost?
- Can we achieve an acceptable reliability by strengthening system components, or should we look at other alternatives?
- If we can't keep a water system operable, what is the most effective means to achieve quick restoration?

Many cities are aware of the concerns. Kobe and San Francisco both have water storage cisterns distributed throughout their urban area. San Francisco has a water system dedicated to fire suppression, while Vancouver, British Columbia, has just placed such a system into service. Berkeley, California is currently designing such a system.

This report helps quantify the problem of water system reliability. It identifies system and component failures in historic events, and summarizes the results. Mitigation measures for system and components are discussed along with other alternatives such as developing alternative water supplies, optimizing system control, and use of GIS as a tool to quicken recovery.

1.1.1 Mutual Needs of Water and Fire Departments

The mutual needs of water and fire departments are discussed in Section 1.2 and 1.3 of Chapter 1.0. Fire departments and water purveyors are in many cases not part of the same governmental jurisdiction. However, fire departments are highly dependent on adequate water supplies to function effectively. Large fires reaching conflagration proportions have become more common. In many instances, water supply to meet large fire flow demands is insufficient, and in many cases, liaison between water agencies and fire departments is poor or ineffective.

In some cases, legal and financial authorization governing the raising of capital to pay for facilities is inconsistent with fire department needs. It is common for state regulations to include requirements for minimum levels of fire flows for new facilities. As fire flow demands increase, it may be difficult for water agencies to legally justify charging rate payers to build facilities to enhance fire flows, and providing for higher fire flows can often lead to a reduction of water quality during normal usage.

Coordination between water and fire agencies must take place during pre-event planning, and should include operational procedures and priorities. Fire departments should attain a good working knowledge of the water system they use, and should understand the earthquake vulnerability of that system.

1.1.2 Earthquakes Hazards

Earthquake hazard concepts and their quantification are described in Chapter 2. Fault rupture results in earthquakes with a measured magnitude, and site-specific intensity. Fault offsets can also cause tsunamis. Ground motion is often measured in terms of peak ground acceleration. Earthquake shaking can result in liquefaction and landslides that can be particularly damaging to buried water system components.

1.1.3 Performance of Water Systems in Historic Earthquakes and Two Other Disasters

Chapter 3 of this report summarizes historic performance of water systems in earthquakes, as well as two instances of non-earthquake-related water system stress events. More detailed accounts from each event are described in Appendix A. The objective is to identify common earthquake deficiencies that repeatedly occur, resulting in dysfunction of water systems for post-earthquake fire suppression and domestic water supply. There are four categories of earthquake and natural hazard disasters described herein:

1. Major urban earthquakes - Kobe, Northridge, and Loma Prieta earthquakes.

2. Less destructive rural or smaller magnitude earthquakes that caused water system damage - Landers/Big Bear, Petrolia, and Whittier earthquakes.

3. Great historic devastating urban earthquakes - Kanto, 1923, and San Francisco, 1906.

4. Potable water system outage from other natural disasters - Des Moines, 1993 flood, 12 days water outage resulting from flooding of a water treatment plant; and, Oakland Hills, 1991 Fire exacerbated by a water supply system originally designed for small fires.

1.1.4 Performance Criteria

Performance criteria are proposed in Chapter 4.0, which ties five categories of water system performance to the probability of earthquake exceedance. Ground motions with a 50 percent chance of exceedance and a 10 percent chance of exceedance in 50 years are set as the earthquake demands. Performance objectives for life safety, fire suppression, public health, system restoration, and property damage are defined for each demand. Each of these five categories are tied to systems and equipment required to meet these performance objectives.

1.1.5 Reliability Analysis

Based on the research of the above events using a system component function analysis, the reliability of municipal water supplies is addressed in Chapter 5.0. The risk of the causative earthquake hazards is quantified. Likely system failure modes are defined. Key system component parameters such as percent of vulnerable pipe materials and percent of area susceptible to liquefaction, that have led to system failure are identified. The performance of mitigation measures that have been subjected to earthquakes are also reviewed. Typical system failures have resulted from:

- Pipe breakage in liquefiable areas resulting in water stored in reservoirs to quickly drain.
- Damage to major transmission lines cutting off the water source.
- Reservoir and connecting piping damage allowing water to drain.
- Loss of power to pump stations making them inoperable.

The results of this project element are the quantification of water system reliability based on seismic risk and key component parameters.

1.1.6 System Component Damage

Key component damage that has resulted in water system failure is researched, and the results described (Chapter 6.0), with earthquake damage clearly defined. Key components include:

- Pipelines
- Reservoirs
- Treatment Plants and Pump Stations

1.1.7 Mitigation Alternatives

Five mitigation alternative strategies to quickly restore water service following an earthquake are described and evaluated in Chapter 7.0. The five mitigation strategies include:

1. Improve communications between Fire and Water Departments.

2. Improve system hardware, such as pipe replacement, reservoir strengthening, and provision of emergency power.

3. Provide post-earthquake system monitoring and control to isolate damaged sections of the system allowing operation of other segments.

4. Develop alternative water sources/supplies such as those dedicated solely to fire protection, or for multi-use.

5. Develop application of geographic information systems, GIS, as a tool to identify areas of water systems vulnerable to earthquakes for use by personnel responsible for fire suppression and engineers responsible for system earthquake mitigation.

Conclusions and recommendations are presented at the end of Chapter 7.0 providing guidance on effective alternatives to increase the reliability of water supply to suppress fires and provide domestic water supply following earthquakes, and a means to provide expeditious restoration of water supply after the earthquake.

1.2 MUTUAL NEEDS OF FIRE AND WATER DEPARTMENTS

Consistently providing adequate water supply for fire protection requires close liaison and cooperation between the fire and water departments. Unfortunately, these agencies are in most cases not part of the same governmental jurisdiction. As a result, understanding and awareness between the organizations can be lacking, particularly when a water utility is quite large and serves a large area. Fire Departments tend to be smaller organizations in size and in many rural areas will be volunteer or part-paid and volunteer. Coordination between the two organizations can be complex and difficult to achieve in such situations.

Fire departments are largely dependent upon adequate water supply in order to function effectively. Fire flow demands can be quite large and in many cases communities that have recently expanded may not have sufficient water main capacity to meet the new water supply demands. Close coordination of developers, planners, the water utility and local fire departments is critical in the planning process. Adequate hydrant distribution must be provided for, gridding and looping of mains is vital to insure adequate fire flows.

As the urbanization of America continues at a high pace, the inter-relationship of water and fire departments becomes more and more important. Large fires, reaching conflagration proportions were at one time limited to industrialized urban areas and cities. However since 1975, it has become commonplace to see major fires in rural and suburban areas due to wild land fires that quickly get out of control. Television brings these dramatic action scenes into our living rooms every year. Other disasters such as earthquakes, train wrecks, arson and civil disturbances can also bring in their wake large fires which may develop into conflagrations.

In many of these major incidents, water supply was insufficient to meet the large demand that was required for control and mitigation. During the event it was often difficult or impossible for water agencies to provide supplies that were required. In many cases liaison between the water agencies and fire departments was poor or ineffective.

Water agencies are tasked with the mission to provide adequate supplies of drinking water for domestic, commercial and industrial usage to serve the populations within their service area. Legally, they are usually required to provide sufficient drinking water for the population, and water supply for fire protection is established by insurance industry standards for fire flow requirements of each community. Lack of sufficient water supply for fire flows can result in higher fire insurance premiums or no fire insurance at all. Communities working with their water districts will usually pay for larger sized mains and increased reservoir storage to allow for fire flow requirements. However, providing large capacity mains for small suburban communities is excessively expensive and can lead to degraded water quality under normal daily usage.

During emergency and disaster response, fire and water departments have mutual needs to maintain fire flows in order to control the event as well as to provide domestic water within the service district. During such times of high stress, water systems will be taxed to their capacity. The interrelationship of fire and water departments comes into play during such critical events. Fire departments must have a good working knowledge of the water supply system for it to maintain continuous operations during emergencies and disasters; they must also be aware of the weak points and deficiencies likely to develop in water supply during emergency operations.

This coordination and knowledge must take place during pre-event planning, actual response and at post-event debriefing.

Pre-event planning should include:

- Identify system vulnerabilities and likely post-earthquake system performance.
- Development of joint Fire/Water department operational policies and procedures (including identification of key personnel and their roles for implementation) to address possible loss or deficient flow rates of water during emergency operations, such as for fire fighting.
- Development of an improved emergency plan for Fire/Water Department operations and liaison at fires and other emergencies.

In addition, Fire Departments should attain a good working knowledge of the water system or systems they will utilize. Hydrant flow rates and pressures, water supply capacity, gate books, maps and other information on water systems should be made readily available. Fire Departments should identify a Water Supply Officer to represent their responsibility in this process.

1.3 EMERGENCY OPERATIONS: WATER SUPPLY NEEDS AND PROBLEMS

Major fires that occur require large water flows to bring them under control. Major cities usually have the required infrastructure to provide the water flows, (8,000-12,000 GPM) in highly congested downtown and industrial districts. However, in residential areas, even city water systems may not have the high capacity, (large mains, pumps, reservoirs), that may be required during a major fire or conflagration that may occur following an earthquake or wild land fire. Current capacities typically range from 500 gpm to 1,500 gpm in smaller suburban areas; however, large fires require fire flows in excess of 10,000 gpm.

Areas of cities or suburban areas that do not have the water supply capacity for conflagrations or major fires face two choices: (1) they must concentrate on reducing the risk of large fire conflagrations (elimination of fuel sources, rapid fire department response while the fire is still small, etc.), or (2) they can upgrade the water system to provide higher reliable flows, such as special connections for emergency hook up by fire departments to water department tanks, reservoirs or access to water shed areas. These emergency sources of supply can allow fire pumpers to connect to a water source even when water mains may be out of service or inadequate main size limits the flows that are needed.

The Oakland Hills fire of October 1991 was a dramatic example of how a water system designed for suburban fire flow rates was overtaxed during a great firestorm. Pre 1930 small diameter water mains prevented access to large water supplies from large reservoirs. Excessive drafting by fire departments led to drawdown rates 300% to 500% above rated capacities. Loss of electric power had essentially little to no effect on the water systems capability to provide supplies to the fire department. A second occurrence in 1993 happened in Malibu, California; wild land fires destroyed hundreds of large homes, many of them within ½ mile of the Pacific Coast highway which has a 30 inch water main. Unfortunately, inadequate mains into the residential areas could not utilize this large supply of water. The fire burned to the Pacific Coast highway in that incident.

2.0 EARTHQUAKE HAZARDS

The purpose of this chapter is to provide an overview of the earthquake concepts and effects that are relevant to the seismic reliability and performance of water supply systems. Section 2.1 discusses basic concepts and describes the seismic hazards that are important to water supply systems. Section 2.2 presents a summary of current methods available to quantify these hazards for single-location facilities, such as tanks, pump stations, or buildings, and for pipelines, which are distributed over large areas.

2.1 EARTHQUAKE CONCEPTS AND HAZARDS

2.1.1 Fault Rupture

An earthquake is generated by sudden movement within the earth that is usually caused by the rupture of a fault. Rupture occurs when the stresses across the fault exceed the strength of the rock comprising the fault. The rupture length along the fault can extend from several meters (microtremors) to several hundred kilometers (great earthquakes) to approximately 1000 km (giant earthquakes). The corresponding displacement at a given location on the fault rupture also increases from a few centimeters for small earthquakes to a few tens of meters for giant earthquakes.

The length of fault rupture and the amount of fault displacement are important parameters for lifelines such as water pipelines. In most of the California urban areas for example, the water transmission and distribution systems invariably cross major faults. Therefore, the identification and characterization of these faults are key components of lifeline hazard and risk assessments and of the seismic design of these systems.

2.1.2 Seismic Moment and Magnitude

The fault-rupture length, width and displacement are one measure of the size of the earthquake. The product of these parameters and the fault rigidity is defined as the seismic moment of an earthquake, so named because the units are (force) x (length). Because it is a measure of the physical size of an earthquake, the seismic moment became the basis for the development in the late 1970s of a new magnitude scale called moment magnitude (M_w), which is now the magnitude scale most commonly used in engineering seismology. For moderate to large earthquakes, the moment magnitude is roughly equivalent to the more traditional magnitudes, such as Gutenberg-Richter ($M_{G\text{-}R}$), local (M_L), body-wave (m_b) and surface-wave (M_s) magnitudes. However, these older magnitude scales cannot distinguish between the great and giant earthquakes because the motions recorded by the seismographs used to compute these magnitudes do not increase further for these big earthquakes (Heaton et al., 1986). This phenomenon of "diminishing returns" in the motion is called saturation. As an illustration, the local magnitude of the great 1906 San Francisco earthquake, which ruptured about 430 km of the San Andreas fault, was 6.9, whereas its moment magnitude was approximately 7.8. The surface-wave magnitude of this event was estimated to be around 8¼. By contrast, the giant 1960 Chilean earthquake, which ruptured approximately 1,000 km of the southern Chile subduction zone, had a similar surface-wave magnitude (~ 8½), but its moment magnitude, which was a superior indicator of the size of this event, was 9.5. This event is the largest earthquake recorded this century. It generated average rupture displacements of approximately 24 m and generated a large tsunami that caused major coastal damage not only in South America, but also around the Pacific Rim and Hawaii.

2.1.3 Intensity

While the moment magnitude scale indirectly measures the physical size of an earthquake, intensity scales measure the effects of the event. Several intensity scales have been developed, but the one most commonly used in the U.S. is the Modified Mercalli Intensity (MMI) scale. To avoid potential confusion with the magnitude scales, values of intensity are given in Roman numerals. The MMI scale runs from I (detected only by sensitive instruments) to XII (damage total). In seismic risk and damage evaluations of water pipeline systems, MMI has been the parameter linking the ground motion with the damage estimates through fragility curves. These curves relate, for example, breaks or spills per length of pipeline to MMI for different types of pipe. The MMI values are typically estimated from the peak ground accelerations (PGA) that are computed along the pipeline route from probabilistic or deterministic seismic hazard analyses.

2.1.4 Ground Motion and Ground Movement

In addition to fault rupture, which can sever buried pipelines and severely damage foundations of structures built in fault zones, earthquake ground motions and earthquake-induced ground failures from soil liquefaction or landslide are significant seismic hazards to water systems. Although buried pipelines have been damaged by vibratory ground motion, aboveground structures such as tanks and buildings are generally more vulnerable to this shaking hazard. Buried pipelines are more vulnerable to permanent ground displacement or deformation (PGD), such as from liquefaction-induced lateral spreading or flow failure of soil, or from landslides. Correlations between PGD and pipeline repair rates have been developed for vulnerability assessments. Nevertheless, even with little or no accompanying lateral soil movement, the soil in a liquefied condition exerts a buoyant force on buried structures (e.g., tanks or pipelines) which can cause these structures, depending on their weight, to rise to the surface or sink further into the soil.

2.1.5 Tsunami and Seiche

Unless aboveground components of a water system are located along the coast or near the edge of a large body of water, tsunamis or seiches pose no threat. Tsunamis are large water waves generated by (1) rapid uplift or subsidence of the seafloor caused by submarine fault rupture, (2) submarine landslide, or (3) volcanic eruption that displaces a large volume of seawater. Tsunamis have caused damage not only to coastal facilities in the epicentral region, but also to facilities several thousands of kilometers away. For example, portions of Hawaii have been devastated by tsunamis originating in Alaska and Chile.

Seiches are standing water waves produced in enclosed or partially enclosed bodies of water such as lakes or, in some cases, harbors. Vibratory motion of the bottom of these bodies of water induce a seiche in the same manner that vibratory ground motion induces sloshing of water within a storage tank.

2.1.6 Hazard from Bridges

Failures of bridge structures carrying pipelines are another potentially serious threat. Older bridges in the epicentral region of a major earthquake, especially those at water crossings where the soil may be more susceptible to liquefaction, are especially vulnerable. Embankment slumping, abutment wall movement or failure, pier foundation settlement or rotation, crushing of concrete columns, and fallen spans are common types of bridge damage that can severely damage a pipeline. Furthermore, pipelines can also be damaged if they are not adequately braced to the bridge superstructure.

2.2 QUANTIFICATION OF HAZARDS

A variety of methods are available to quantify the various seismic hazards discussed in Section 2.1. These methods range from simple empirical approaches to complex numerical modeling. Methods that have been applied to quantify seismic hazards as part of a seismic risk study of water systems are briefly summarized in the following subsections.

2.2.1 Surface Fault Rupture and Magnitude

Considerable amounts of observational data have been collected on the amount and type of surface fault rupture during historical earthquakes. Clear correlations are observed between earthquake magnitude and surface-rupture parameters such as surface-rupture length (SRL), maximum surface-rupture displacement (MD), and average surface-rupture displacement over the length of fault rupture (AD). Many statistical correlations among these parameters have been developed over the last 40 years. The most recent set by Wells and Coppersmith (1994) is based on source-parameter data from 244 worldwide shallow crustal earthquakes. These authors conducted statistical regression analyses of these data to produce correlations between M_w, SRL, MD , AD and other source parameters. The mathematical form of these correlations (using M_w and SRL as an example) is:

$$M_w = a + b \log (SRL) \tag{2.1}$$

$$\log (SRL) = c + d\, M_w \tag{2.2}$$

where a, b, c and d are regression coefficients from a least-squares analyses.

The first formula is used to predict the magnitude given the surface rupture length, and the second formula is used to predict the surface rupture length given the magnitude. Because of the manner in which the regressions are performed, the two formulas are not equivalent unless the M_w - SRL data are perfectly correlated. This nonequivalence means that a given SRL substituted into the first formula will yield a magnitude, which when substituted into the second formula, yields a different SRL. The correlations of interest to risk assessments of water systems are M_w vs. SRL , SRL Vs M_w , MD Vs. M_w , and AD Vs. M_w , where the first variable listed in each correlation is the dependent variable to be predicted. The first correlation (M_w Vs. SRL) is used to estimate the maximum magnitude earthquake a particular fault can generate. This magnitude is used in the probabilistic and/or deterministic calculations of ground motions for the system risk evaluation or design. The other three correlations (SRL Vs. M_w , MD Vs. M_w , AD Vs. M_w) are primarily useful for risk assessment or design of pipelines crossing active faults.

2.2.2 Ground Motion

For most earthquake hazard or risk evaluations, ground motions are computed from simple equations (called attenuation equations) that express the ground-motion parameter of interest as a function of magnitude, distance from the fault rupture to a location where the ground motion is to be computed, and usually other variables, such as site geology and fault mechanism. The ground-motion parameters relevant to buried pipeline response are peak ground acceleration and peak ground velocity, which are directly related to flexural and axial pipeline strain through simple models (ASCE, 1984). The peak ground acceleration is also used to assess liquefaction and landslide potential.

Peak ground acceleration and response spectra are the ground-motion parameters typically used in the evaluation for aboveground structures. For damage estimates, the peak ground acceleration has typically been converted to MMI, as noted in the previous section. However, new methods are being developed to estimate damage from response spectra.

Most attenuation equations applied to facilities in the western U.S. were derived from a statistical regression analyses of ground accelerations recorded during mostly California earthquakes. For the eastern U.S. where ground-motion data are lacking, attenuation equations have been derived mainly from seismological models of the earthquake source and travel paths of the seismic waves; most of these models have been calibrated against the limited recorded data. A convenient summary of many of the new attenuation equations is provided in Seismological Research Letters (Jan/Feb., 1997, Vol. 68, No. 1) published by the Seismological Society of America.

Attenuation equations are used directly to estimate ground motions for regional scenario earthquakes, such as a M_w 8.0 for the Southern San Andreas fault or M_w 7.5 for the Wasatch fault. Such approaches are often called deterministic. Attenuation equations are also input to probabilistic approaches that compute the probabilities of exceeding given levels of ground motion in some time period. Output from these probabilistic methods are then input to probabilistic risk methodologies that compute annual probability of exceeding given levels of damage, typically expressed as dollar loss.

These probabalistic approaches to estimate ground-motion have been recently used by the U.S. Geological Survey (USGS) to generate maps of the ground-motion hazard throughout the U.S. (Frankel et al., 1996). These maps, published in 1996 and available on the Internet, show contours of bedrock spectral acceleration that have a 2% probability of being exceeded in a 50 year period; these maps are intended for incorporation into the 1997 National Earthquake Hazard Reduction Program (NEHRP) Seismic Provisions. The previous maps showed spectral accelerations with a 10% probability of being exceeded in 50 years. The reason for adopting a lower probability for the new maps was to account for the large infrequent earthquakes outside California that have recurrence intervals on the order of several thousand years. Such events would have a much greater impact on the ground motions with 2% in 50 year probability (which corresponds to an average frequency of approximately once every 2500 years) than on the ground motions with a 10% in 50 year probability (average frequency of approximately once every 500 years). Because they affect much larger areas with the capacity to inflict greater damage on a per area basis, the large infrequent earthquakes must be considered in risk evaluations also.

2.2.3 Intensity

For risk evaluations, MMI is often the parameter that links the ground motion with the damage estimates through fragility curves, as previously explained. MMI is usually estimated from simple correlations between MMI and peak ground acceleration (PGA). A common form of the relationship used in practice is

$$\mathrm{MMI} = a + b \log(\mathrm{PGA}) \qquad (2.3)$$

where a and b are constants. Typically, the correlations in the literature are expressed as

$$\log(\mathrm{PGA}) = c + d\,(\mathrm{MMI}) \qquad (2.4)$$

where the constants, c and d, are derived by regression analyses of MMI - PGA data recorded during past earthquakes. For a listing of some of these relationships, see Trifunac and Brady (1975), Murphy and O'Brien (1978), and O'Brien et al (1976). For application to risk analysis, an appropriate equation is selected and solved algebraically in the form of Eqn (2.3). Strictly speaking, the constants, c and d, will be different than those derived directly from regression of Eqn (2.3) on the data. Unfortunately, these regressions were not performed; however, the algebraic solution has been considered an acceptable approximation.

2.2.4 Liquefaction and Landslide

The methods for liquefaction and landslide evaluations vary in complexity from simple empirical approaches to complex finite element or finite difference modeling. For a critical facility at a single location, such as a reservoir or tank, both approaches and some intermediary ones have been used in stability evaluations. For distributed systems, such as pipelines, empirical based approaches are preferred because they can be implemented much more easily and quickly over a broad area. Empirical based methods for liquefaction assessment generally include correlations between cyclic stress ratio and field Standard Penetration Test (SPT) or Cone Penetration Test (CPT) measurements that are dependent on earthquake magnitude and the fines content of the soil. These simple methods define the potential of soil liquefaction for a given magnitude and ground acceleration. This liquefaction potential can be expressed as a simple "yes-liquefaction will occur" or "no-liquefaction will not occur" with a computed safety factor greater or less than 1.0, or the liquefaction potential can be expressed as a conditional probability given the earthquake magnitude and ground acceleration.

If only geologic maps are available instead of SPT or CPT data, which is often the case for existing pipelines, then a liquefaction susceptibility of each mapped geologic unit is first determined. For example, high susceptibility is usually assigned to late Holocene stream deposits, while low susceptibility is assigned early Pleistocene deposits. Tabular correlations between these liquefaction susceptibilities and MMI are then established that define the liquefaction potential, which is usually stated as a probability that the unit will liquefy given the MMI. If large areas such as river valleys or plains are located in a high seismic region, then this simple procedure will predict that the entire area will have a high liquefaction potential. Observations from past earthquakes demonstrate that only a fraction of the total area experiencing strong shaking shows any visible signs of liquefaction. Thus, modification factors have been introduced in liquefaction risk analysis to account for this phenomenon.

Simple empirical based methods for landslide susceptibility involve the mapping of geologic units and slope angles, and developing correlations between these variables and landslide susceptibility. Because historic or prehistoric landslides are often easy to recognize, field observations are important data for assessing landslide susceptibility.

The amount of permanent ground deformation from liquefaction or landslide can be estimated using (1) simple empirical equations, such as those developed by Bartlett and Youd (1995), (2) simple mechanical models, such as the sliding block model by Newmark (1965), and (3) complex nonlinear numerical analysis codes capable of computing permanent deformation. Bartlett and Youd's equations are used to compute the lateral spread displacement for a sloping ground surface or a free face condition such as a river bank. The Newmark sliding block model is used to estimate the total movement of a soil mass sliding down a slope subjected to a given ground acceleration time history. Several nonlinear finite element and finite difference codes compute permanent ground deformations (settlements, slumping, lateral spreading). Although these codes have often been used for stability assessments of earth dams, they have also been applied to structures such as water tanks supported on unstable ground.

2.2.5 Tsunami and Seiche

Although most coastal areas facing the Pacific Ocean are susceptible to tsunamis, the estimation of the likelihood of damage to coastal facilities is complicated by the fact that the tsunami could originate from a local offshore earthquake or from a distant earthquake more than several thousand kilometers away. In addition to its size, the damage potential of a tsunami is greatly influenced by the bathemetry and topography of the coastal areas. These important factors affect the amount of water runup (or inundation). which is the major cause of tsunami damage. For the estimation of damage, historical data of tsunami runup can be useful as well as results from numerical models. Nonetheless, considerable uncertainty exists in estimating probabilities of tsunami damage and judgment is necessary for such evaluations. Judgment is also required for similar evaluations of seiches.

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3.0 PERFORMANCE OF WATER SYSTEMS IN HISTORIC EARTHQUAKES

3.1 INTRODUCTION

The section summarizes historic performance of water systems in earthquakes, as well as two instances of non-earthquake-related water system failure. Detailed accounts from each event are described in Appendix A. The objective is to identify common earthquake deficiencies that repeatedly occur, resulting in disfunction of water systems for post-earthquake fire suppression and domestic water supply. There are four categories of earthquake and natural hazard disasters summarized herein:

1. Major urban earthquakes – 1995 Kobe, 1994 Northridge, and 1989 Loma Prieta earthquakes.

2. Less destructive rural or smaller magnitude earthquakes that caused water system damage – 1992 Landers/Big Bear, 1992 Petrolia, and 1987 Whittier earthquakes.

3. Great historic devastating urban earthquakes – 1923 Kanto, and 1906 San Francisco.

4. Potable water system outage from other natural disasters – 1993 Des Moines flood, 12 days water outage resulting from flooding of a water treatment plant; and, 1991 Oakland Hills fire, exacerbated by inadequate water supply.

The summary is organized by consequences of water system failure and system component failure that caused system disfunction, rather than disaster events. Refer to Table 3-1 for a numerical summary; the explanatory text is provided below in Section 3.2 and 3.3. The numbers within this table are ratings; these ratings are shown in parentheses in the explanatory text. Note that these ratings are based on the descriptions included in Appendix A, and do not include performance of similar facilities in different jurisdictions in the same earthquake, or other earthquakes.

3.2 FAILURE CONSEQUENCES

Disaster Event Year – (Column 1)

Fire Suppression/Lacked Water Supply – (Column 2)

Lack of water for fire suppression limited fire suppression if there were fires in all of these cases. A rating of 5 indicates complete, wide spread water system disruption. A rating of 3 indicates limited water system disruption in limited areas.

Fire – (Column 3)

This column identifies whether there was a fire, and the significance of the fire. Major fires (5), or conflagrations, occurred in the Kobe, Kanto, and San Francisco earthquakes. In the Loma Prieta and Northridge earthquakes, there were significant fires (4), but were generally limited to a single block each. In the Petrolia earthquake, one strip mall burned, and in Des Moines flood, one industry burned (3). In both Whittier and Landers/Big Bear earthquakes, there was loss of water service, but there were no ignitions reported (1).

Used Alternate Supply – (Column 4)

A rating of 1 indicates aggressive, successful use of alternate supplies with the following examples. In the Loma Prieta earthquake, the portable water supply system was used to pump water from San

Francisco Bay, and pump it to the fire. In the Northridge earthquake, water was pumped from swimming pools. The Des Moines flood is rated as a 2, where tankers had been brought in to use in case of a fire, and were used to haul water to an industrial fire. In the Oakland Hills fire, a very limited portable water supply system was set up late in the response. A rating of 3 indicates use of alternate supplies with moderate success with the following examples. In the Kobe earthquake, water was pumped from cisterns which were quickly drained; they were able to set a pump system from Osaka Bay many hours after the event, but many fire grounds were located too far away from the bay for this to be effective. In the Whittier earthquake, tankers normally used for grass fires were relocated, but were not needed. In the Kanto earthquake (5), there were access points to Tokyo Bay and numerous rivers and inlets. Apparently available equipment did not allow pumping from these supplies. San Francisco's fire pumps did not have the capability to draft from the bay in 1906 (5). Alternate supplies were not mentioned in the other events.

Cooling Telephone Central Offices and Computers – (Column 5)

In both the Northridge earthquake, and the Des Moines flood, tank trucks hauled in water to keep telephone central office air conditioners operating to keep computer switches cool. Also in Des Moines, the Principal Insurance Company brought in water by tank truck to keep air conditioners operable to keep computers functioning (4).

3.3 SYSTEM COMPONENT FAILURES

Surface Supply Failure – (Column 6)

In the Kanto earthquake, landslides covered Tokyo's river intakes (4). In the San Francisco earthquake, there was damage to supply impoundment dams, but they did not fail (2).

Raw Water Transmission – (Column 7)

In the Northridge, Kobe, Kanto, and San Francisco earthquakes, the raw water transmission line(s) failed, significantly impacting water system function (5). In the Loma Prieta earthquake, one of EBMUD's treatment plants was rendered inoperable for several days as a result of the failure of the raw water line. They were able to serve the area with alternate supplies (3).

WTP Damage – (Column 8)

The Des Moines water treatment plant (WTP) and finished water pump station were submerged in the flood. This submergence was the primary cause of system failure (5). Treatment plants were damaged in the Northridge, Kobe, and Kanto earthquakes, but probably were not the primary cause of system failure (3).

Well Casing/Equipment Failure – (Column 9)

Wells were exposed in the Landers, Whittier, and Kanto earthquakes, but with no apparent damage to the casing or equipment (1).

Loss of Power – (Column 10)

Loss of power was a significant issue that required portable pumping/generator equipment to be brought in during the Northridge earthquake. Loss of power was a minor issue in the Oakland Hills fire. It was not the controlling factor (2). In the Kobe, Landers/Big Bear, and Petrolia earthquakes, power loss kept wells and pump stations from functioning, but other parts of the systems were heavily damaged, and loss of power had little significance (3).

Tank Damage, Shell/Structure – (Column 11)

Tank shells suffered elephant's foot buckling, and in a few cases, split in the Northridge, Landers/Big Bear, and Whittier earthquakes. These failures are not thought to have been the primary cause for system failure (3).

Tank Inlet/Outlet Pipe – (Column 12)

Tank inlet/outlet and drain pipes failed with major (5) or significant (4) system effects in the Northridge, and the Landers/Big Bear and Whittier earthquakes, respectively. The literature indicates that pipe connection failures were much more common, resulting in much greater impact on system performance than were tank structural failures. There was only one tank pipe inlet/outlet failure in Kobe (2).

Pipe Damage, Permanent Ground Deformation, (PGD), (i.e. lateral spread, landslide, fault offset) and Wave Propagation – (Columns 13 & 14)

Pipeline failures due to PGD had a major impact on system performance in every earthquake (5). Question marks, ?, following the rating identify ratings where the literature is unclear about soil conditions where pipe damage occurred. In the Kobe, Northridge, and Landers/Big Bear earthquakes, there was significant pipe damage where there was no PGD. In the Loma Prieta and San Francisco earthquakes, the PGD was specifically identified as a controlling parameter in several areas. There was only one pipe failure in the Des Moines Flood, likely due to scour (2).

Building Services – (Column 15)

Service failures were identified as placing a heavy hydraulic demand on the system in the Kobe, Kanto, and San Francisco earthquakes, and the Oakland Hills Fire (5 rating). In Kanto, San Francisco, Oakland Hills, and to a lesser degree, Kobe, service failures resulted when building structures burned down, and connecting piping failed and began to leak. Services were sheared off in the Marina District in the Loma Prieta Earthquake (3).

3.4 CONCLUSIONS

There is a correlation between incidents where there was inadequate water for fire suppression, and where fire became a significant issue. The only events where fire was not an issue were Whittier, which was a relatively small earthquake, and Landers, where there is a sparse population and building density. There is also a strong correlation between the three most significant fires, San Francisco in 1906, Kanto, and Kobe, and the ineffective use/availability of an alternate water supply.

The following list groups system component failures that caused system dysfunction into four priority ratings. Pipeline damage had the greatest impact in most of the events.

Very High

1. Pipe Damage – Permanent Ground Deformation (PGD)

High

2. Raw Water Transmission (Pipeline)
3. Pipe Damage - Wave Propagation

Moderate

4. Loss of Power
5. Water Treatment Plant Damage
6. Tank Damage - Inlet/Outlet Pipe Damage

Low

7. Tank Damage - Shell/Structure Damage
8. Surface Supply Failure
9. Well Casing/Equipment Damage

TABLE 3-1
SUMMARY - PERFORMANCE OF WATER SYSTEMS IN EARTHQUAKES

Disaster Event		Failure Consequences				System Component Failure									
						Supply/Treatment					Tank		Pipe		
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
	Year	Fire Suppression/ Lacked Water Supply	Fire	Used Alt. Supply	Cooling Telephone CO's and Computers	Surface Supply Failure	Raw Water Transmission	WTP Damage	Well Casing/Equipment Damage	Loss of Power	Shell/Structure	Inlet/Outlet Pipe	PGD (lateral spread, landslide, fault offset)	Wave Propagation	Building Services
EARTHQUAKE															
San Francisco	1906	5	5	5	NA	2	5	NA	NA	1	1	1	5	3	5
Kanto	1923	5	5	5	NA	4	5	3	1	1	1	1	5?	3?	5
Whittier	1987	3	1	3	NA	1	1	1	1	4	3	4	5?	3?	1
Loma Prieta (EBMUD only)	1989	4	4	1	1	1	3	1	NA	1	1	NA	5	3	3
Landers/Big Bear	1992	5	1	NA	NA	1	NA	NA	1	3	3	4	5	5	1
Petrolia	1992	5	3	1	NA	1	NA	NA	NA	3?	1	1	5?	3?	1
Northridge	1994	5	4	5	4	1	5	3	NA	4	3	5	5	5	1
Kobe	1995	5	5	3	?	1	5	3	NA	3	1	2	5	5	5
OTHER DISASTERS															
Oakland Hills Fire	1991	4	5	2	NA	NA	NA	1	NA	2	NA	NA	NA	NA	5
Des Moines Flood	1993	5	3	2	4	1	1	5	NA	4	1	1	2	1	1
AVERAGE		4.6	3.6	3.0	3.0	1.4	3.6	2.4	1.0	2.6	1.7	2.4	4.7	3.4	2.8

NOTE: Unless otherwise noted in the text, a rating of 1 means no reported problem.

4.0 PERFORMANCE CRITERIA

Post-earthquake operational goals and objectives are defined for water systems to help prioritize facility upgrades. These operational goals and objectives are defined for two levels of earthquake ground motion.

Operating Basis Earthquake (OBE) ground motions are defined to have a 50% chance of exceedance in 50 years (equivalent to an average return interval of 72 years). Design Basis Earthquake (DBE) ground motions are defined to have a 10% chance of exceedance in 50 years (equivalent to an average return interval of 475 years). Thus, OBE ground motions are likely within the typical lifetime (50 years) of a civil facility, while the DBE ground motions are not likely to occur within the facility's lifetime. Nonetheless, the probability of a DBE event is high enough to be considered a credible scenario.

The system objectives cover source, treatment, distribution and storage subsystems.

4.1 PERFORMANCE CATEGORIES

There are five performance categories that are applicable to water and wastewater systems. Performance objectives, prioritized in order of importance, can be categorized to include:

1. Life Safety - prevention of conditions that can result in injury or loss of life
2. Fire Suppression - ability to supply water to suppress fires (especially critical after an earthquake)
3. Public Health - prevention of disease transmission by providing uncontaminated drinking water
4. System Restoration/Loss of Business Opportunity - being able to provide water service to commercial, industrial and residential customers so that businesses can operate
5. Property Damage - prevention of direct damage to infrastructure or damage to private facilities caused by failure of infrastructure.

4.2 POST-EARTHQUAKE PERFORMANCE OBJECTIVES

Earthquake performance objectives are defined as the desired performance for selected levels of earthquake intensity. Earthquake performance can be expressed in terms of the consequences that result from given states of system performance. Typical post-earthquake adverse consequences that result from water system and/or component failure are presented in Table 4-1.

The recommended acceptable adverse consequence levels for water are shown in Table 4-2. The recommendations are based on the assumption that the system is existing, and that strengthening all functions of the system to sustain no damage in an OBE and only minimal damage in a DBE would be prohibitively expensive.

4.3 SYSTEM, SUBSYSTEM AND COMPONENT EARTHQUAKE PERFORMANCE CONSEQUENCES AND UPGRADE PRIORITIZATION

The severity of adverse consequences is related to system performance. For example, if there was significant pipeline damage, tanks would quickly drain, and impair fire suppression capabilities.

Typical failure consequences, expressed in terms of the performance categories of generic water system components, are presented in Table 4-3. For each defined subsystem, the damage scenarios are presented for progressively increasing damage levels. By comparing the adverse consequences with the objectives, acceptable and unacceptable subsystem performance can be delineated. Performance that violates the Table 4-2 objectives (which are repeated at the top of Tables 4-3) are shaded.

In addition to delineating acceptable and unacceptable consequences, Table 4-3 can be used to help prioritize components for upgrade. For example, damage at a spring supply that does not affect functionality or life safety may violate the property damage objective for an OBE scenario. However, reservoir drainage may violate fire suppression, system restoration and property damage for OBE and DBE objectives. Because reservoir drainage violates more objectives and the fire suppression objective is considered more important than the property damage objective violated by the spring, upgrade of the reservoir to prevent drainage would be considered higher priority than upgrade of a spring to prevent damage that does not affect facility functionality or life safety.

Violation of the performance objectives presented in Table 4-2, along with the objective importance can be used to help prioritize vulnerable components for upgrade. Other factors that will be considered in prioritization for upgrade will include long term capital improvement (e.g., it does not make sense to upgrade a component that will soon be replaced), redundancy and upgrade cost-effectiveness.

In many instances, system performance and achievement of the performance objectives will be determined by collective component performance and not by individual component performance. For example in the water system, failure of transmission piping would stop most of the system supply, but water for fire suppression could be maintained by water stored in the system (if the reservoirs did not fail), meeting the fire suppression earthquake objectives in Table 4-1.

TABLE 4-1. POST-EARTHQUAKE WATER SYSTEM/COMPONENT FAILURE CONSEQUENCES

PERFORMANCE CATEGORY	TYPICAL ADVERSE CONSEQUENCES
Life Safety	Chlorine release or building collapse
Fire Suppression	Unavailability of water to fight fires that results in increased risk of life safety and property loss.
Public Health	Unavailability of potable water safe to drink (drinking water can be delivered in tank trucks or bottled).
System Restoration	Unavailability of water for domestic, commercial, and industrial uses. Includes secondary losses from business interruption, caused by unavailability of water for manufacturing purposes, fire sprinkler systems, food preparation, sanitation at public facilities and agricultural irrigation. Also includes loss of revenue from water utility's inability to sell water.
Property Damage	Repair and replacement of water utility infrastructure damaged by ground shaking or permanent ground displacement. May also include indirect damage from debris impact, flooding, erosion, or undermining to water utility infrastructure or private facilities near failed water utility facilities.

TABLE 4-2. WATER PERFORMANCE OBJECTIVES - ACCEPTABLE ADVERSE CONSEQUENCE LEVELS FOR TWO EARTHQUAKE LEVELS

PERFORMANCE CATEGORY	ACCEPTABLE ADVERSE CONSEQUENCES	
	OBE (50% chance in 50 years)	DBE (10% chance in 50 years)
Life Safety	Negligible - Loss of life is not an acceptable consequence, but minor injuries may occur	Minimal - Loss of life is not an acceptable consequence, but injuries are expected
Fire Suppression	Minimal – With the exception of small isolated areas that are not densely populated, water for fire suppression should be available for entire service area.	Moderate - Water for fire suppression should be available for a minimum of 70% of the service area. All industrial areas and densely populated business and residential areas should have water available for fire suppression (possibly from area lakes and rivers).
Public Health	Low – Water should be available for all but a few isolated areas. Boil water order acceptable for up to 48 hours.	Moderate - Service should be available for at least 50% of system. Boil water order, delivery by tanker truck, or bottled water acceptable for up to one week. Restoration to 100% service within one week.
System Restoration	Low – Water should be available for all but a few isolated areas.	Moderate - Service should be available for at least 50% of system. Restoration to 100% service within one week.
Property Damage	Low – Any damage should not affect facility functionality and should be repairable. Facilities not owned by the water utility should not be damaged (flooding, debris impact, etc.) by utility facility damage	Moderate - Complete loss of nonessential facilities acceptable if it is not cost-effective to upgrade them and other performance objectives are not violated. Critical facilities not owned by the water utility (e.g., communications facilities, hospitals, etc.) should not be damaged (flooding, debris impact, etc.) by utility facility damage.

TABLE 4-3. TYPICAL WATER SUBSYSTEM DAMAGE ADVERSE CONSEQUENCE LEVELS

SUBSYSTEM PERFORMANCE	PERFORMANCE CATEGORY				
	LIFE SAFETY	FIRE SUPPRESSION	PUBLIC HEALTH	SYSTEM RESTORATION	PROPERTY DAMAGE
OBE Consequence Level Objective	Negligible	Minimal	Low	Low	Low
DBE Consequence Level Objective	Minimal	Moderate	Moderate	Moderate	Moderate
Sources (Intakes, Wells, Springs, Treatment Plants, Etc.)					
- No damage	None	None	None	None	None
- Nonstructural and/or structural damage that is repairable and does not affect facility functionality or life safety	None	None	None	None	Moderate
- Nonstructural and/or structural damage that leads to loss of functionality but does not affect life safety	None	Moderate	Moderate	High	High
- Nonstructural and/or structural damage that leads to life safety hazards as well as loss of functionality (e.g., collapse or hazardous material release)	High	Moderate	Moderate	High	High
Transmission and Distribution Piping					
- No damage	None	None	None	None	None
- Joint damage resulting in a few leaks in transmission or large (e.g., larger than 10-inches in diameter) distribution lines and/or a few breaks in smaller (e.g., less than 10-inches in diameter) distribution line that cause loss of hydraulic continuity	None	Minimal	Low	Low	Low
- Transmission or large distribution line failure and/or widespread damage in distribution piping	None	High	Moderate	High	High
Pump Stations					
- No damage	None	None	None	None	None
- Nonstructural and/or structural damage that is repairable and does not affect facility functionality or life safety	None	None	None	None	Moderate
- Nonstructural and/or structural damage that leads to loss of functionality but does not affect life safety	None	Moderate	Low	Moderate	Moderate
- Nonstructural and/or structural damage that leads to life safety hazards (e.g., collapse or hazardous material release)	High	Moderate	Low	Moderate	High

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TABLE 4-3. TYPICAL WATER SUBSYSTEM DAMAGE ADVERSE CONSEQUENCE LEVELS (CONTINUED)

SUBSYSTEM PERFORMANCE	PERFORMANCE CATEGORY				
	LIFE SAFETY	FIRE SUPPRESSION	PUBLIC HEALTH	SYSTEM RESTORATION	PROPERTY DAMAGE
OBE Consequence Level Objective	Negligible	Minimal	Low	Low	Low
DBE Consequence Level Objective	Minimal	Moderate	Moderate	Moderate	Moderate
Reservoirs					
- No damage	None	None	None	None	None
- Miscellaneous damage but no loss of contents	None	None	None	None	Low
- Tank drainage caused by attached piping failure or tank structural damage	Minimal	High	Low	Moderate	High
- Elevated tank collapse	Moderate	High	Low	Moderate	High

5.0 WATER SYSTEM RELIABILITY

There are several types of approaches that can be used to assess post-earthquake water system reliability. The traditional method is to perform deterministic assessments of each water system component and use the component assessment results to develop a system performance scenario. Another approach is to express component vulnerability in probabilistic terms and use probabilistic techniques to evaluate system reliability. An approach that has been used in other industries but that has not yet been used extensively in the water industry is to use system reliability assessment techniques such as fault tree analysis. The increasing accessibility of GIS data bases and software allows presentation of system assessment results in geographic formats that can be easily used and interpreted by planners, emergency response personnel and engineers.

5.1 DETERMINISTIC ASSESSMENT OF WATER SYSTEM RELIABILITY

The most common approach to assess water system reliability is to use deterministic methods. Typically, three steps are involved in a deterministic approach. First, the seismic hazards are defined. Based on the seismic hazards and component characteristics, vulnerability component, seismic risk is then determined. The final step is to use the component seismic risk to predict overall system performance.

5.1.1 Earthquake Hazard Development

Varying levels of detail can be used to assess earthquake hazards. In a deterministic approach, hazards are typically defined for discrete scenarios.

Discrete scenarios may be defined in terms of earthquake size and location for a specific earthquake source zone or fault. For a water district in Northern California, a scenario earthquake might be defined as a M8.0 earthquake that occurs on the San Andreas Fault with an epicenter postulated at a given location. Based on this event, attenuation relationships may used to estimate ground motion levels in terms of peak ground acceleration or peak ground velocity for different facility sites. Seismic hazard models can also be used to estimate permanent ground displacements from liquefaction/lateral spread, landslide and fault rupture throughout the area of concern. Example models include Joyner and Boore (1981) (peak horizontal acceleration and velocity), Bartlett and Youd (1995) (liquefaction/lateral spread displacements), Newmark (1965) (landslide displacements), and Wells and Coppersmith (1994) (fault rupture displacements). See Chapter 2.0 for a discussion of earthquake hazards.

Another approach is to use discrete generic scenarios. Many codes and standards define ground motions in probabilistic terms. The Uniform Building Code (UBC) (International Conference of Building Officials, 1994) and NEHRP Provisions define design levels with ground motions that have a 10% chance of exceedance in 50 years (equivalent to a 475 year average return interval). The NEHRP (Building Seismic Safety Council, 1994) ground acceleration map is shown in Figure 5-1. Ground shaking intensity can be defined from acceleration levels. For many urban areas in the western United States, maps that show permanent ground displacements have also been developed. A map that shows permanent ground displacements for the South Seattle Quadrangle from Mabey and Youd (1991) is shown in Figure 5-2.

If more accuracy is required, earthquake hazards can also be assessed on a site-specific basis instead of using regional maps or macrozonation techniques. The disadvantage of evaluating hazards on a site-specific basis is that more detailed geologic information and more detailed analyses are required.

Consequently, site-specific seismic hazard assessments are usually only performed for more critical water system facilities and/or facilities chosen for detailed analysis or upgrade design.

5.1.2 Component Vulnerability Assessment

Once the seismic hazards have been determined, the next step in a water system reliability assessment is to perform vulnerability assessments for the water system components and facilities. Building and civil structure component vulnerability assessments can be performed by using rapid visual screening techniques or by more sophisticated analyses. In many instances, rapid visual screening procedures are used to identify potentially vulnerable components and facilities for more detailed analysis. For pipelines, a preliminary assessment can be performed by identifying pipe material/joint types that are the most vulnerable, and identify whether they are subject to PGD.

5.1.2.1 Rapid Visual Screening

Rapid visual screening techniques can, with minimal resources, be used to quickly identify components and facilities that may be seismically vulnerable. Steps involved in the rapid visual screening technique typically include brief design document review (if the design documents are available) and visual inspection of the component or facility. In some instances, simple calculations or analyses may be performed.

Design documents are reviewed to determine the seismic design criteria, component/facility characteristics (particularly those that may not be evident from a visual inspection) and structural properties. A visual inspection is made to identify features that may not be shown in the design documents and determine the state-of-repair. Checklists or evaluation statements (e.g., see Building Seismic Safety Council, 1992) may be used to help identify vulnerable features during the design document review or visual inspection. In some instance, simple calculations can be performed to verify component or member seismic adequacy.

Vulnerability can be based on the seismic design criteria, identification of features or characteristics that may have performed exceptionally well (or poorly) in previous earthquakes (e.g. Ballantyne, 1994), and the results of simple analyses. More rigorous analyses may be performed in a later phase for those components found to be vulnerable or marginally vulnerable.

5.1.2.2 Analytical Techniques

More complex analyses procedures are usually only conducted for critical facilities and/or for facilities that have been identified to be potentially vulnerable during the rapid visual screening phase. Analytical techniques can range from code and standard compliance checks to highly sophisticated finite element analyses. Analytical techniques for various types of water system facilities are discussed in more detail in Section 6.

5.1.3 System Performance

Given the vulnerability of a water system's components and facilities, overall system performance can be determined. A simple assessment can be made by using each facility's vulnerability to predict whether the facility will be functional for the defined earthquake scenario. Then, based on the systems characteristics, engineering judgment can be used to assess how loss of vulnerable facilities will affect system performance.

An alternative approach to engineering judgment is to use network analyses techniques to more rigorously assess system performance. The most feasible approach is to use software available in the public domain designed particularly for water system network analysis.

In Kennedy/Jenks/Chilton (1990), KYPIPE, a commercially available hydraulic network analysis program, was used to model water availability in portions of the Seattle Water Department's service area. Assessments were conducted for three hypothetical earthquake scenarios. Results for one of the earthquake scenarios are shown in Figure 5-3.

5.2 PROBABILISTIC ASSESSMENT OF POST-EARTHQUAKE WATER SYSTEM RELIABILITY

There is a significant amount of uncertainty associated with earthquake hazards and the response of facilities subjected to earthquake hazards. Although accurately modeling this uncertainty is difficult, probabilistic assessments can be used to assess the magnitude and likelihood of variations from the expected outcome (e.g., Ang and Tang, 1975).

5.2.1 Earthquake Hazard and Component Vulnerability Models

Earthquake ground shaking levels are determined by probabilistically combining the ground motions from all potential earthquake sources that can affect a site. Uncertainty in earthquake ground shaking is typically expressed in terms of probability of exceedance or average return interval. Similarly, component vulnerability may be expressed as a function in terms of failure probability versus ground shaking level.

5.2.2 System Functionality

Once the earthquake hazards and component vulnerabilities (fragilities) have been defined, system functionality can be predicted. Sometimes, a discrete earthquake event is chosen and only the uncertainties in the component vulnerabilities are considered. For example, the earthquake hazards associated with a 0.10 probability of exceedance in 50 years may be determined. The probabilities of component failure are then obtained from the fragility curves.

By using Monte Carlo simulation techniques in conjunction with hydraulic models, functionality can be expressed in probabilistic terms for different locations in the system. If functionality expressed in probabilistic terms on a yearly (or other unit of time) basis is needed, the uncertainty associated with the earthquake hazards can be included in the Monte Carlo simulation.

5.3 SYSTEM RELIABILITY ASSESSMENT USING FAULT TREES

Although fault trees have been used extensively in many industries such as the nuclear industry, fault trees have not been used as extensively by water utilities. Fault trees can be used to calculate failure probabilities, identify paths that may lead to failure, and identify those events that are most likely to lead to failure. Detailed discussion on fault tree analysis can be found in such references as Ang and Tang (1984) or Russell et. al. (1994, Volume 1).

5.3.1 Fault Tree Development

In this subsection, a basic fault tree is developed for water systems as they relate to earthquake reliability. For any particular location within a water system, two top events were defined:

- Loss of Water Pressure for Fire Suppression
- No Drinking Water Supply (Loss of pressure or contamination)

The fault tree for post-earthquake water system reliability is presented as Figure 5-4. The tree shows that the top event is "inadequate water." The tree was developed so the same tree structure can be used to assess the probability of inadequate water for fire suppression or inadequate drinking water supply. The events that can lead to inadequate water for fire suppression are an independent subset of the events that can lead to inadequate drinking water supply. Although the same tree structure can be used for fire suppression water or drinking water, a separate analysis is needed to assess these events.

The fault tree was also constructed to keep events as independent as reasonable. By keeping the events independent, it is easier to deal with events that may not be relevant for a particular location in the water system service area. For example, the location may be served by gravity without any reliance on pumping. Because the loss of pumping capability events are independent of other events that may lead to loss of pressure, the fault tree can still be used by simply assigning event probabilities of 0.0 to those events related to pumping failures.

Note that the fault tree logic shown in Figure 5-4 may need to be modified to fit the system characteristics of different locations within a water system and for different water systems. Additionally, many of the basic events shown in Figure 5-4 can be further developed.

5.3.2 Application of Fault Trees to Water System Seismic Reliability Assessment

Fault trees are used to determine the probability of occurrence of a top event. It is possible for water systems to suffer partial failure. That is, one area (pressure zone) of the system may be functional while another area may be without pressure. Consequently, fault tree analysis is most applicable to pressure zone reliability assessment within a water system. However, if a logical definition of water system failure is provided, fault trees can be used to assess overall water system performance.

The most significant advantages of using fault trees for water systems are:

- Because fault trees are probabilistically based, uncertainty from earthquake hazard severity can be incorporated into the assessment. In a deterministic approach, the assessment must be performed for a discrete scenario. In a probabilistic assessment, a discrete scenario can be assessed or the probabilities of all possible scenarios can be convoluted. For example, the probability that a pressure zone will lose water in a 25 year period can be determined.
- In addition to identifying the series of events (cut sets) that are the most significant contributors to the occurrence of the top event, fault trees can be used easily to perform sensitivity analyses. These features make fault tree analysis a valuable tool for loss estimation and cost/benefit studies.

Significant disadvantages of using fault trees for water system reliability assessment include:

- It is difficult to determine occurrence probabilities for basic events that can lead to water system failure from earthquakes. Sophisticated seismic data bases and analyses of these data bases has not been performed for many water system components.
- Water system and pressure zone performance is time dependent. A separate analysis would need to be performed for each time of concern.
- Because water system performance may vary by area, a separate assessment is needed for each area.

5.3.3 Fault Tree Analysis Demonstration for Post Earthquake Water System Reliability Assessment

The fault tree shown in Figure 5-4 is used to assess the post-earthquake reliability of water supply at Point A for the hypothetical water system shown in Figure 5-5. The system is assumed to be located in a moderately seismic region of the United States such as the Pacific Northwest, Wasatch Front (Utah) or New Madrid (Midwest) area where large but infrequent earthquakes are possible. Until recently, seismic hazards have not been considered in the design of water system facilities in many of these areas. Consequently, it is assumed that many of the system facilities are vulnerable. The basic event definitions and their assumed probabilities are presented in Table 5-1.

Using the computer program IRRAS (Russell et. al., 1994, Volume 2), the upper bound on the probability of no water pressure for fire suppression at Point A in the water system was calculated to be 0.9998. In Figure 5-6, the relative importance of each basic event is shown. Figure 5-6 indicates that developing the alternate source (ESOURCE1) would have the most impact (highest Risk Reduction Ratio) of any single event on reducing the likelihood of not having fire suppression water at Point A.

Assuming that the alternate source could be developed so it had a reliability of 0.95 (failure probability of 0.05), the fault tree analysis was re-run. However, even by developing this alternate source, the failure probability is still 0.9889.

Because developing another source is very expensive and the benefits appear to be minimal, less expensive alternatives are considered. Figure 5-6 suggests that the next four events that most significantly affect reliability are:

- Failure of the tank inlet/outlet line (ESTORAGE1)
- Tank structural failure leading to tank drainage (ESTORAGE2)
- Excessive distribution pipe breakage (EPIPES4)
- Inability to isolate distribution pipe breakage (EPIPES3)

Because it would be very expensive to replace the distribution piping with more seismic resistant piping, this option is not performed. Consequently, the tank upgrades are performed and valving is added so that the vulnerable distribution piping can quickly be isolated. If the reliability associated with these events is assumed to be 0.95, the probability of being unable to deliver water decreases to 0.47.

Nonstructural upgrades such as anchoring and/or bracing equipment, piping, etc. can be performed at nominal cost. Structural upgrades of simple water system facility buildings, such as pump station and well shelters are also usually inexpensive. Consequently, the following upgrades are performed so the reliability is increased to 0.95 for each event:

- Nonstructural upgrade at the well (EWELL4)
- Improvement of the emergency power system at the intake/source (ESURFACE7)
- Nonstructural upgrade at the pump station (EPUMP2)
- Improvement of the emergency power system at the well (EWELL9)
- Structural upgrade at the well (EWELL3)
- Improvement of the emergency power system at the pump station (EPUMP5)
- Nonstructural upgrade at the intake/source (ESURFACE2)
- Structural upgrade at the pump station (EPUMP1)

By performing these additional upgrades, the probability of not having water pressure at Point A falls to 0.21.

This fault tree demonstration shows the following:

- The methodology for how a fault tree assessment can be used to determine post-earthquake reliability for a water system. The fault tree used in this assessment will likely need to be adjusted to reflect characteristics of different water systems.
- The basic steps on how fault tree analyses can be used to prioritize and/or develop a mitigation program. Because water delivery is typically dependent upon a series of facilities, upgrade of only a single facility will not significantly improve overall reliability if other facilities are also vulnerable.
- It is unrealistic to make water systems completely "earthquake proof". Because it is prohibitively expensive to upgrade some water system facilities, those facilities that are upgraded cannot be made completely reliable, and other events that lead to water unavailability may be beyond the control of the water purveyor. As a result the failure probability can never be completely eliminated and will usually be significant (e.g., greater than 0.05).

5.4 WATER SYSTEM RELIABILITY ASSESSMENT USING GIS

Another tool for water system reliability assessment is the use of Geographic Information System (GIS) technology. GIS software packages can be used to assess the seismic vulnerability of water system facilities and/or to geographically display facility vulnerability and water pipeline water pressure.

5.4.1 Uses of GIS Technology for Seismic Vulnerability Assessment of Water Systems

Although GIS technology has only recently gained widespread use, it has already been applied in several instances for water system seismic assessment. For example, in Heubach (1996) pipeline construction materials and joint types were electronically overlaid on the earthquake hazards. Using Arc Macro Language (a programming language for ArcInfo GIS), damage algorithms that related pipe damage to ground shaking intensity and permanent ground displacement were used to determine pipe damage and vulnerability. GIS was then used to geographically display pipe vulnerability.

By incorporating a hydraulic analysis with a GIS-based assessment, water pressure availability can be geographically displayed (for example, see Hwang and Lin, 1997). The geographic display of expected water availability can be invaluable to fire fighting personnel and emergency preparedness and response planners. In addition to assessing the vulnerability of a current system, the assessment can be used to assess the effectiveness of potential upgrades.

5.4.2 Simplified Methodology for GIS-Based Water System Reliability Assessment

Loss mitigation strategy effectiveness is a function of the reliability of the present system and the reliability of the upgraded system. In this subsection, a GIS-based methodology that can be used to easily assess the effect of system improvements is outlined and demonstrated. This methodology includes five steps:

1. Estimation of seismic hazards such as ground shaking intensity and permanent ground displacement. Models such as those developed by Joyner and Boore (1981), and Bartlett and Youd (1995) can be used to estimate the seismic hazards.

2. Vulnerability assessment of concentrated facilities such as pump stations, treatment plants, sources, etc. These assessments can be performed external to GIS. Alternately, vulnerability algorithms that relate functionality to seismic hazards (e.g., ground shaking intensity and permanent ground displacement) can be evaluated by GIS programming languages.
3. Vulnerability assessment of buried pipelines. The GIS-based approach outlined in Heubach (1995) is used for the demonstration assessment. Note that for key transmission lines, a facility-specific assessment based on structural mechanics theory may be more appropriate than the use of empirical damage algorithms.
4. Development of a simplified system hydraulic model. In the simplified hydraulic model, each pressure zone is modeled as single node (as opposed to a network of pipes). Concentrated facilities are also modeled with nodes. Single links are used to connect nodes that are connected by pipelines.
5. Hydraulic analysis of the system and geographical display of the results. In some GIS programming languages, it may be possible to call existing network analysis programs as external subroutines.

Hwang and Lin (1997) used a similar approach with GIS to assess the Memphis Light, Gas and Water system in Shelby County, Tennessee. The procedure demonstrated in this subsection is similar to the procedure used in Hwang and Lin. However, in order to greatly reduce the complexity of the hydraulic analysis, the hydraulic model is greatly simplified from the models used by Hwang and Lin or by Kennedy/Jenks/Chilton. Although the EPANET (Rossman, 1994) hydraulic model was run externally to the GIS program for this demonstration, it is possible to use EPANET as an external subprogram. By setting up EPANET as part of the GIS assessment, it would be possible to perform "near" real-time assessments of a water system after an earthquake and use these assessments to prioritize repair of damaged facilities.

5.4.3 Demonstration of Simplified GIS Methodology

The simplified model of the system used for demonstration purposes is shown in Figure 5-7. This system consists of the following facilities:

- Four pressure zones: the 250, 350, 550 and 600 zones
- Two springs: Spring 11 (7000 gallon per minute maximum capacity) and Spring 12 (5000 gallon per minute maximum capacity)
- One well field/treatment facility: Well 13 (4000 gallon per minute maximum capacity)
- Seven reservoirs: Reservoir 31 (6 million gallons), Reservoir 32 (3.5 million gallons), Reservoir 33 (1 million gallons), Reservoir 34 (6 million gallons), Reservoir 35 (3 million gallons), Reservoir 36 (1 million gallons) and Reservoir 37 (300,000 gallons)

5.4.3.1 Description of Demonstration System

The demonstration water system is supplied primarily from three source: the two springs located outside the service area and a third well field within the service area. Water from the well field must be treated. Except during the peak summer demand season, the springs can serve the 250 and 600 pressure zones by gravity. Should the transmission line from Spring No. 11 fail, an intertie at Spring No. 12 could still transport water from Spring No. 11 to the service area. However, the intertie will not permit gravity flow of water from Spring No. 12 to the service area via the Spring 11 transmission line. A six million gallon reservoir (Reservoir 31) serves as terminal storage for the springs.

The well field is run primarily during the summer peak demand season and supplies water to the 250 pressure zone. The 250 pressure zone is located in an alluvial valley with a high water table. This area is highly susceptible to liquefaction. Water is pumped from the 250 pressure zone up to the 350 and 550 pressure zone.

The 250 pressure zone, which contains primarily commercial and industrial businesses, has the largest water demands. The 550 pressure zone is the second largest zone and serves residential areas and small businesses. The other pressure zones are primarily residential and have smaller water demands than the 250 zone and 550 zone.

5.4.3.2 Demonstration System Hydraulic Model

The demonstration water system is modeled as a network of pipeline links and storage, demand and supply nodes. Pumping capacity is modeled by adjusting the pipeline diameters and lengths to account for pump capacity and head loss.

Each pressure zone is modeled to a single node. Demand for each node is computed as the sum of expected post-earthquake demand for normal potable uses such as drinking and sanitation, post-earthquake fire flow demand, and water loss due to pipe breakage. Potable and fire-suppression demands are treated as user defined inputs for each pressure zone. Pipe breakage was calculated using the GIS-based procedure demonstrated in Heubach (1996). Water loss due to pipe breakage was estimated within the GIS by a procedure similar to the one used by Kennedy/Jenks/Chilton (1990). The total post-earthquake demands for an Operating Basis Earthquake (50% probability of exceedance in 50 years) are as follows:

- 250 Zone – 55,000 gallons per minute
- 350 Zone – 2,000 gallons per minute
- 550 Zone – 1,500 gallons per minute
- 600 Zone – 5,000 gallons per minute

Four different cases were run for an operating basis earthquake (OBE) scenario:

- Case I: All concentrated facilities and transmission pipelines remain functional and distribution piping failures in the 250 zone *cannot* be quickly isolated.
- Case II: All concentrated facilities and transmission pipelines remained functional and distribution piping failures within the 250 pressure zone *are* quickly isolated.
- Case III: Reservoirs Nos. 31 and 36 are not functional and distribution piping failures in the 250 zone *cannot* be quickly isolated.
- Case IV: Reservoirs Nos. 31 and 36 are not functional and distribution piping failures within the 250 pressure zone *are* quickly isolated.

5.4.3.3 Demonstration System Post-Earthquake Water Pressures and Upgrade Recommendations

Water pressure versus time (after the earthquake) for each pressure zone are shown for Cases I, II, III and IV in Figures 5-8, 5-9, 5-10 and 5-11, respectively. The results demonstrate that even if concentrated facilities are fully functional, pipeline breakage can result in complete loss of pressure within hours in an OBE scenario.

Although a design basis earthquake case was not hydraulically modeled, pipe breakage in the 250 zone would likely be so widespread that almost instantaneous loss of water pressure would occur in the 250 zone. Pipe damage in the 600 zone, coupled with demand from the 250 zone, would likely result in complete loss of pressure in the 600 zone also unless the 600 zone was isolated from the 250 zone. Because Reservoirs Nos. 36 and 37 would likely not be functional, water would only be available for a very limited time after a design basis earthquake in the 350 and 550 zones.

The results indicate that upgrade of Reservoir No. 36 (because it has a 1 million gallon capacity as compared to Reservoir No. 37's 300,000 gallon capacity) may be the highest priority upgrade in terms of increasing system reliability and cost-effectiveness. If the pump station that pumps from the 350 zone to the 550 zone is vulnerable, it should also be upgraded. However, upgrade of Reservoir No. 36 would not increase reliability for the 250 and 600 zones. Even with upgrade of Reservoir No. 33, reliability of the 550 zone is not significantly increased.

Because replacing piping within the 250 zone is prohibitively expensive, adding the capability to quickly isolate the 250 zone appears to be the best way to increase the reliability of the 600 zone. However, note that if the 250 zone were isolated, water pressure would likely be lost in the 250 zone much faster because it could not be supplied from the 600 zone. Additionally, the approach of using isolation devices has several drawbacks (e.g., cost, potential to inadvertently isolate areas where water is needed for fire suppression, etc.) and has not yet been proven.

Regardless of whether or not the 250 zone is isolated, the assessment shows that water pressure is likely to be lost in the 250 zone. Consequently, alternative sources should be identified for fire suppression and drinking water.

This demonstration also shows how water system functionality can be modeled using GIS. Although the simplified model does not give (in terms of breakdown of water pressure by grid areas) results as detailed as those obtained by using more complex hydraulic modeling of the system, the simplified methodology is much easier to implement. Because the hydraulic models are simple, input files could be developed within a GIS framework. By developing the models within GIS, this simplified GIS-based water system assessment could be performed as part of a seismic mitigation program to assess system vulnerability and/or assess the merits of upgrades on post-earthquake system performance. Additionally, if the assessment is performed entirely within GIS, the methodology could be used as a tool in post-earthquake recovery efforts to prioritize facility restoration so that system recovery is optimized.

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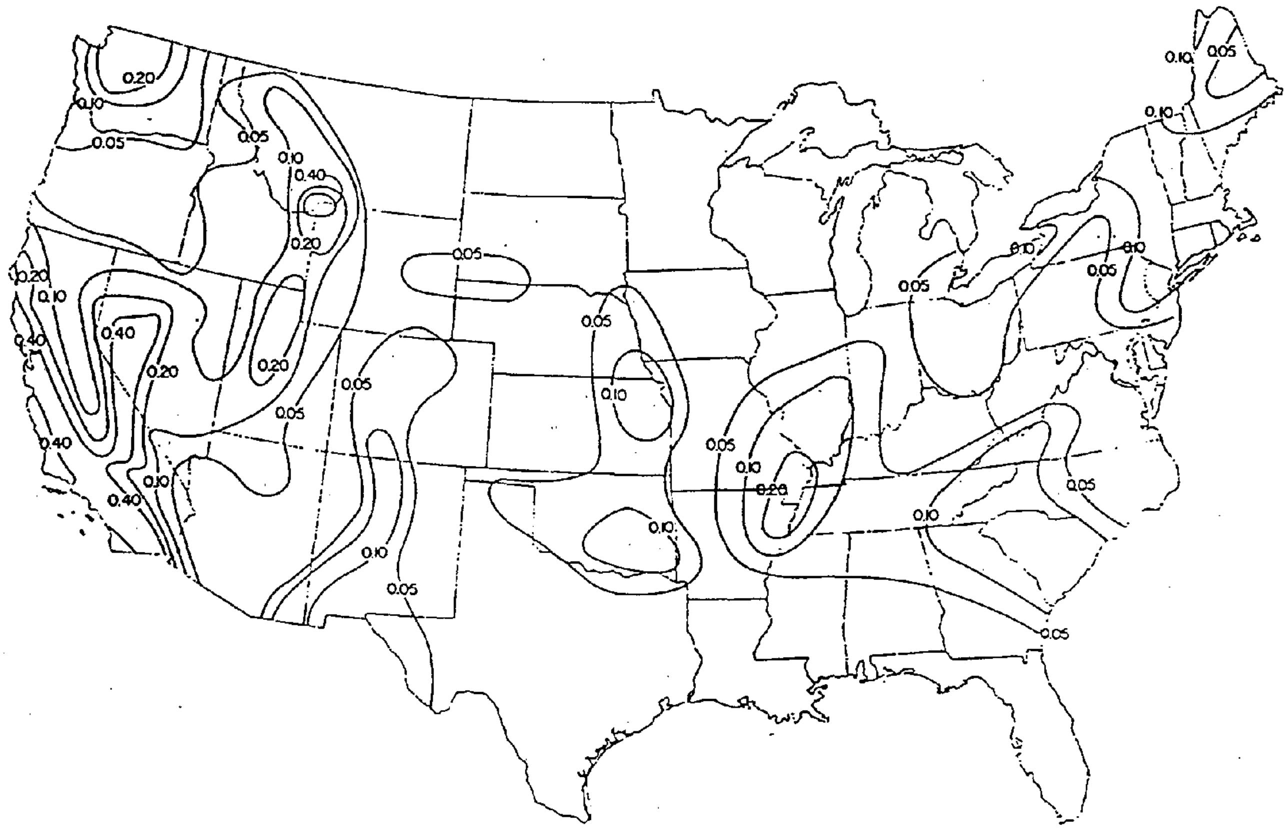
TABLE 5-1 BASIC EVENT FAILURE PROBABILITIES

EVENT ID	DESCRIPTION	FAILURE PROBABILITY
ESOURCE1	Alternative source not available	1.0
ESOURCE2	Transmission pipeline from well fails	0.1
ESOURCE3	Surface water transmission pipeline fails	0.8
ESURFACE1	Surface water facility structural failure	0.01
ESURFACE2	Surface water facility nonstructural failure	0.35
ESURFACE3	Permanent ground displacement blocks intake	0.15
ESURFACE4	Source contaminated by man-made contaminant	0.05
ESURFACE5	Source contaminated by natural contaminant	0.65
ESURFACE6	Loss of commercial power at source	0.95
ESURFACE7	Emergency power unavailable at source	0.75
EWELL1	Aquifer stops producing	0.1
EWELL2	Sanding of well occurs	0.15
EWELL3	Well structure (housing) fails	0.2
EWELL4	Nonstructural failure at well site	0.45
EWELL5	Indirect hazard damages well site	0.01
EWELL6	Well casing is sheared or severely bent	0.01
EWELL7	Differential displacement between casing and slab	0.05
EWELL8	Loss of commercial power at well site	0.95
EWELL9	Emergency power is unavailable at well site	0.25
EWELL10	Aquifer is contaminated	0.05
EWELL11	Well head is flooded by contaminated water	0.01
EWELL12	Well head is not properly sealed	0.01
EPUMP1	Pump structure (housing) fails	0.2
EPUMP2	Nonstructural failure at pump station	0.75
EPUMP3	Indirect hazard damages pump station	0.05
EPUMP4	Loss of commercial power at pump station	0.95
EPUMP5	Emergency power is unavailable at pump station	0.55
ESTORAGE1	Inlet/outlet pipe failure	0.05
ESTORAGE2	Tank structural failure	0.05
EPIPES2	Local pipe break isolates area	0.05
EPIPES3	Excessive pipe breakage	0.85
EPIPES4	Pipe breaks are not isolated	0.05
EWATER1	Excessive water demand	0.01

TABLE 5-2 SUMMARY OF CONCENTRATED FACILITY EXPECTED POST-EARTHQUAKE PERFORMANCE

FACILITY	EXPECTED PERFORMANCE	
	OBE	DBE
Spring No. 11	Functional	Functional
Spring No. 12	Functional	Functional
Well Field No. 13/Treatment Plant	Functional	Functional
Tank No. 31	Functional	Functional
Tank No. 32	Functional	Functional
Tank No. 33	Functional	Fails
Tank No. 34	Functional	Functional
Tank No. 35	Functional	Functional
Tank No. 36	Functional	Fails
Tank No. 37	Functional	Fails
Spring No. 11 Transmission Pipeline	Functional	Fails



Note that the numbers on the contours are values of EPA in units of acceleration gravity. They were used to prepare Map 1 in Chapter 1 of the *Provisions*.

Figure 5-1
NEHRP Ground Acceleration Map
(Adopted from Building Seismic Safety Council, 1994)

MAXIMUM LATERAL SPREAD DISPLACEMENT HAZARD FOR SOUTH SEATTLE

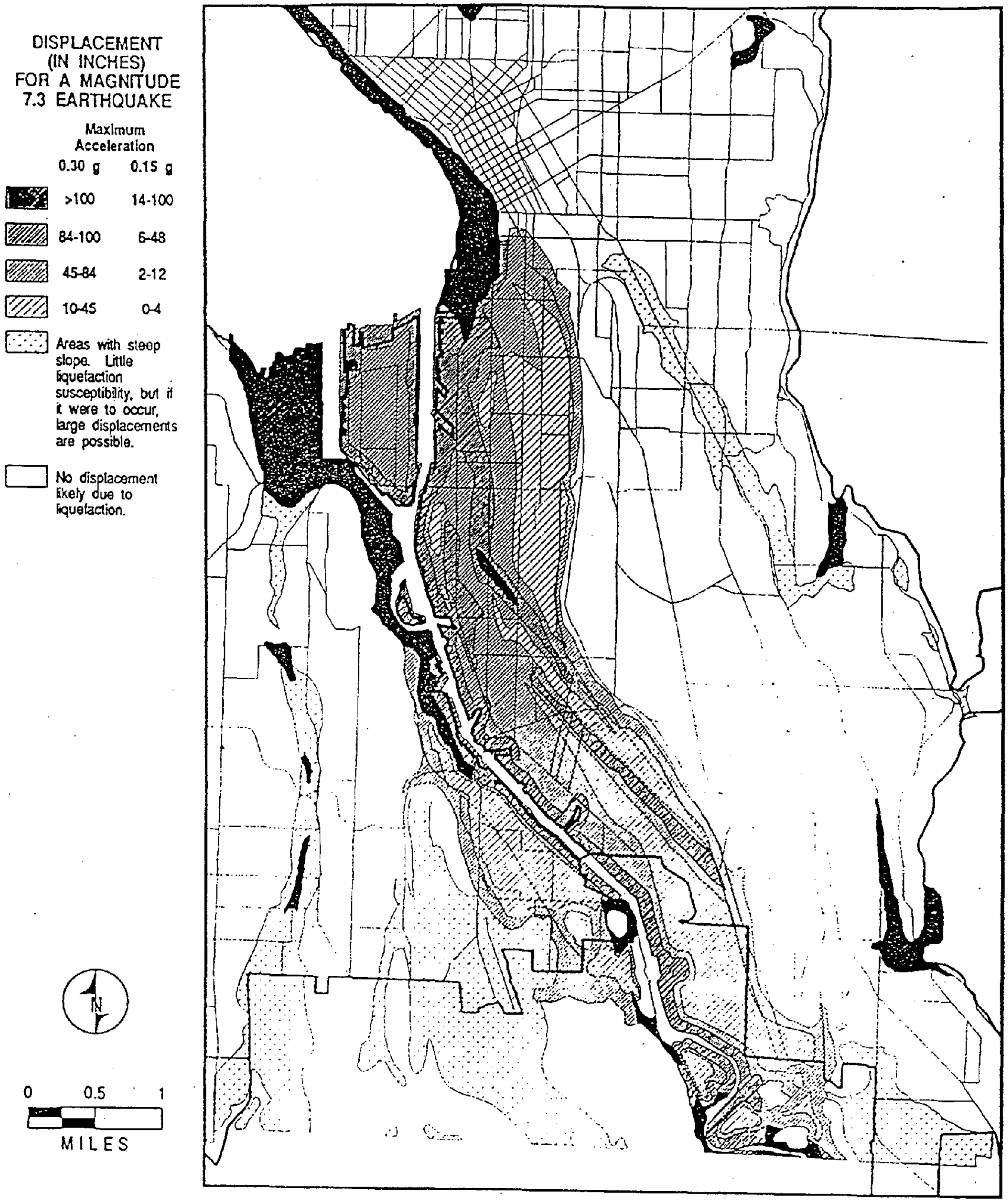


Figure 5-2
Lateral Spread Displacement for the South Seattle Quadrangle
(Adopted from Mabey and Youd, 1991)

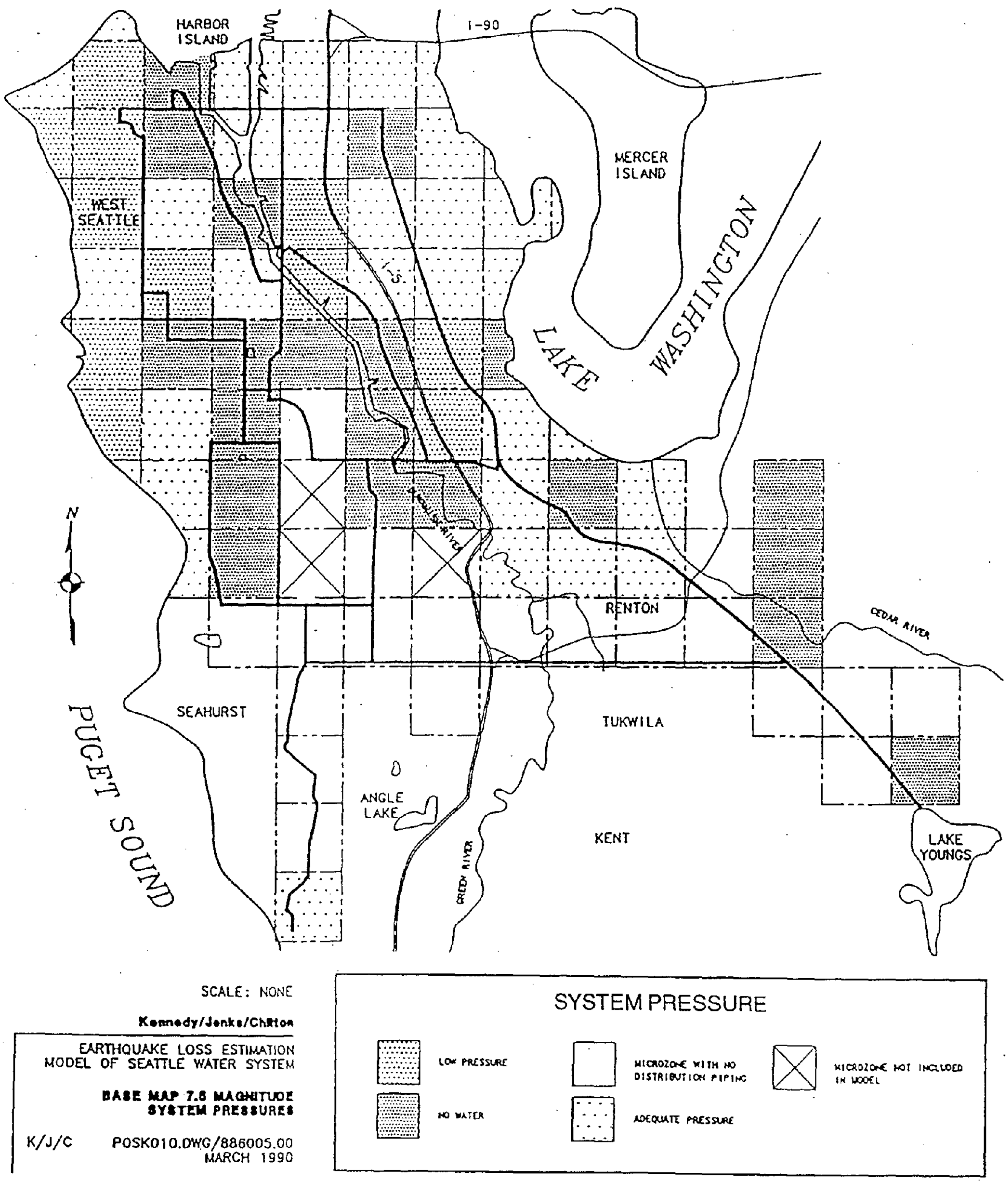


Figure 5-3
Seattle Water System Pressure Following a Hypothetical Earthquake
(Adopted from Kennedy/Jenks/Chilton, 1990)

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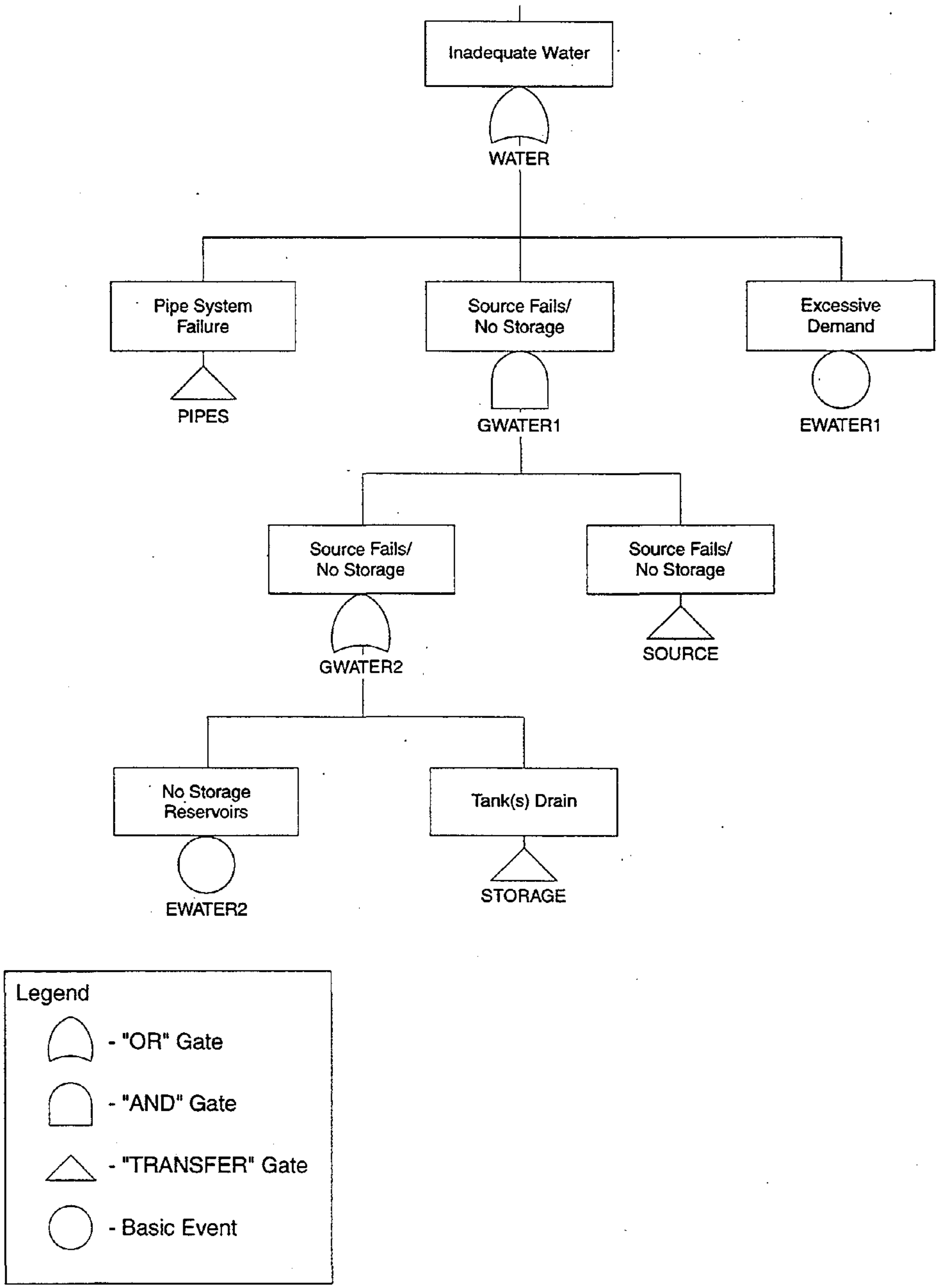

Figure 5-4
Water System Fault Tree (Page 1 of 8)

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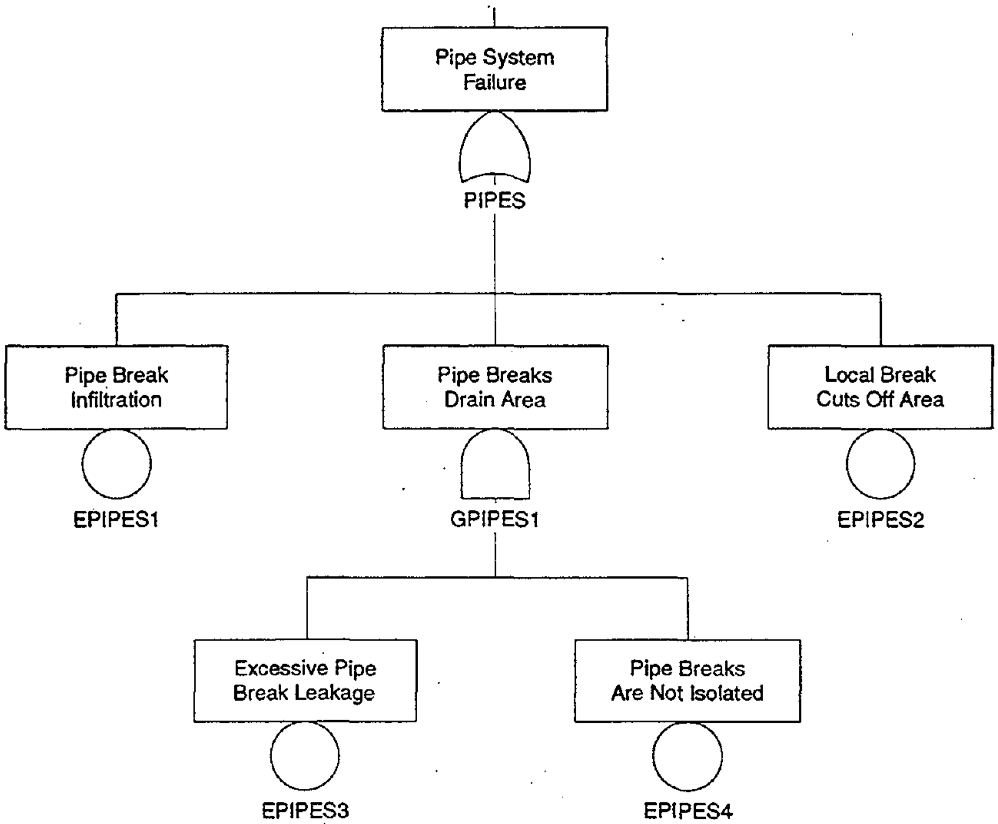

Figure 5-4
Water System Fault Tree (Page 2 of 8)

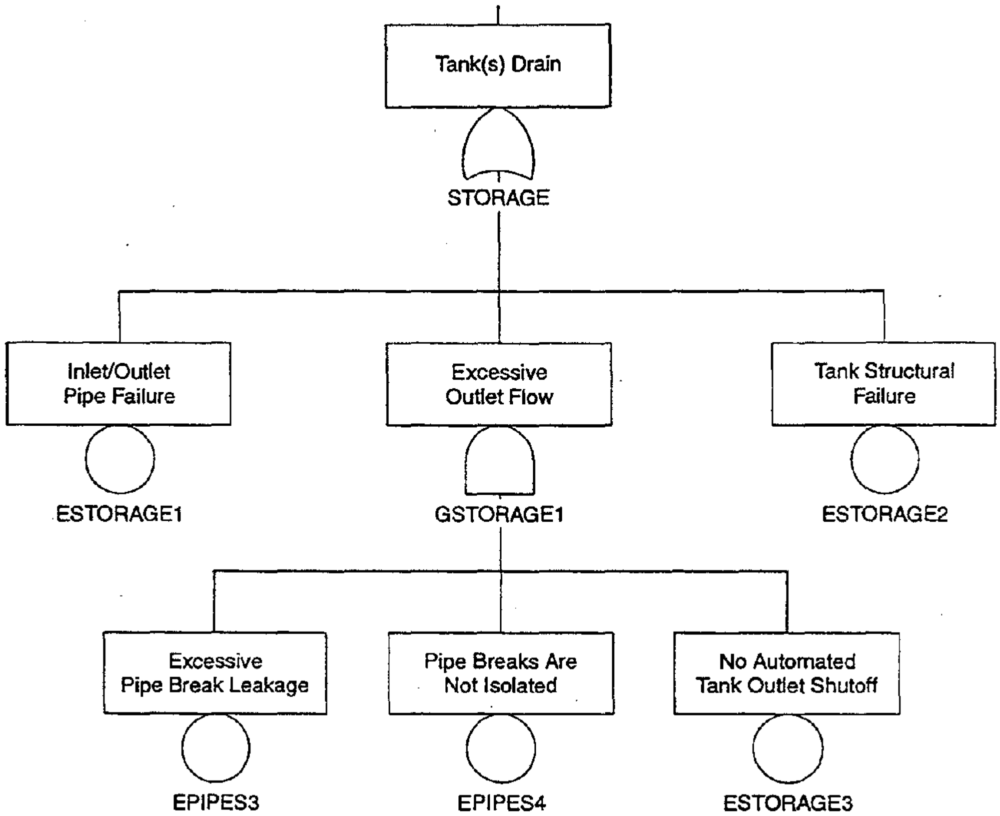

Figure 5-4
Water System Fault Tree (Page 3 of 8)

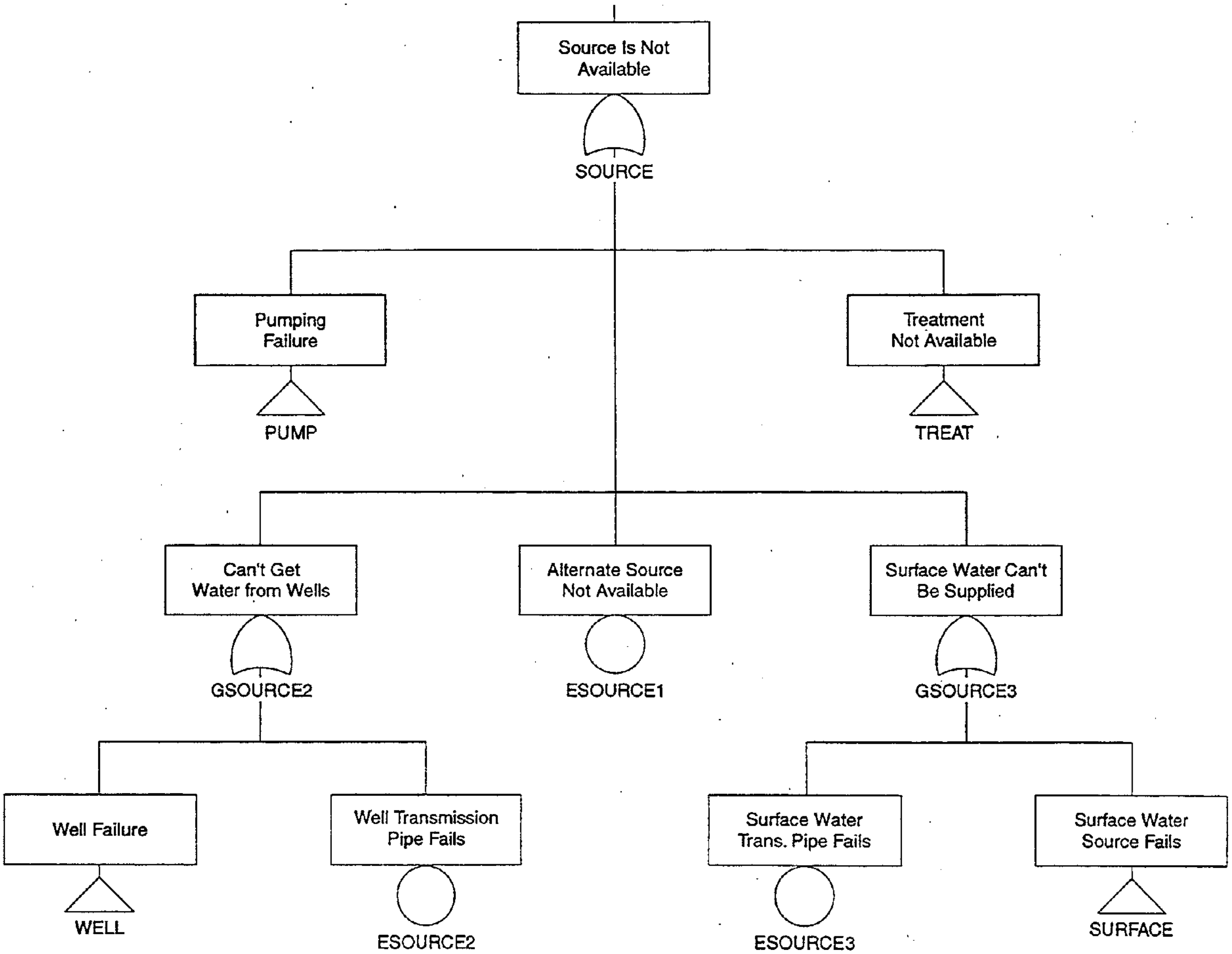

Figure 5-4
Water System Fault Tree (Page 4 of 8)

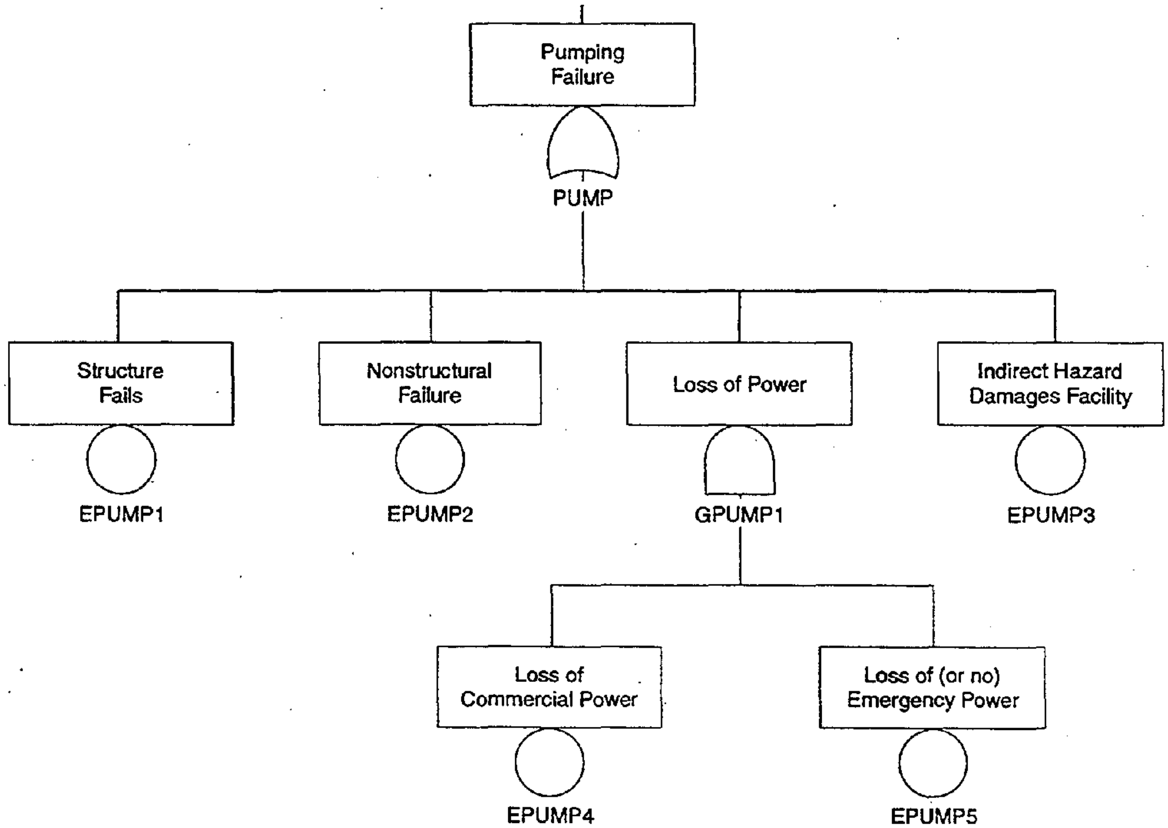

Figure 5-4
Water System Fault Tree (Page 5 of 8)

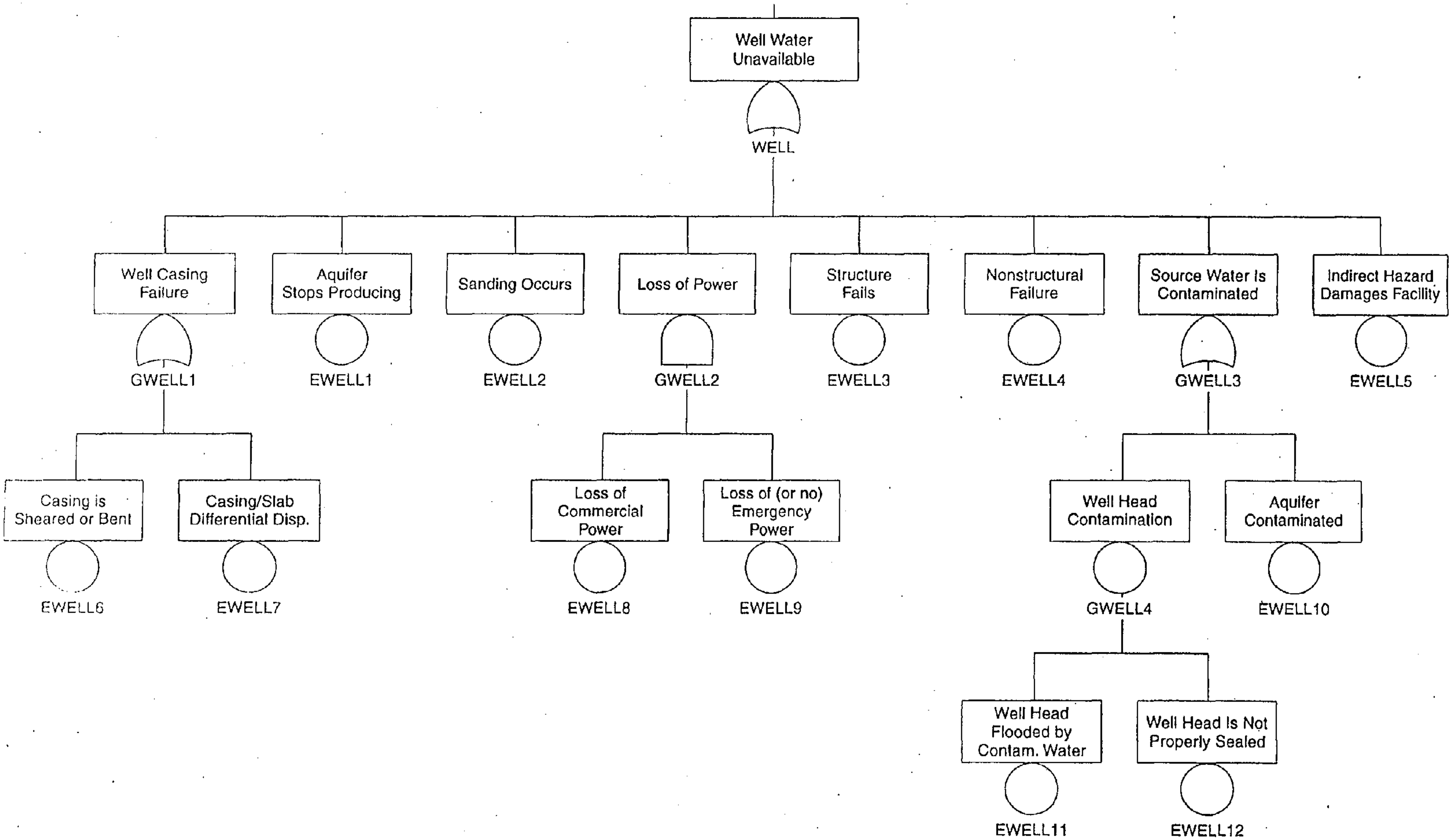

Figure 5-4
Water System Fault Tree (Page 6 of 8)

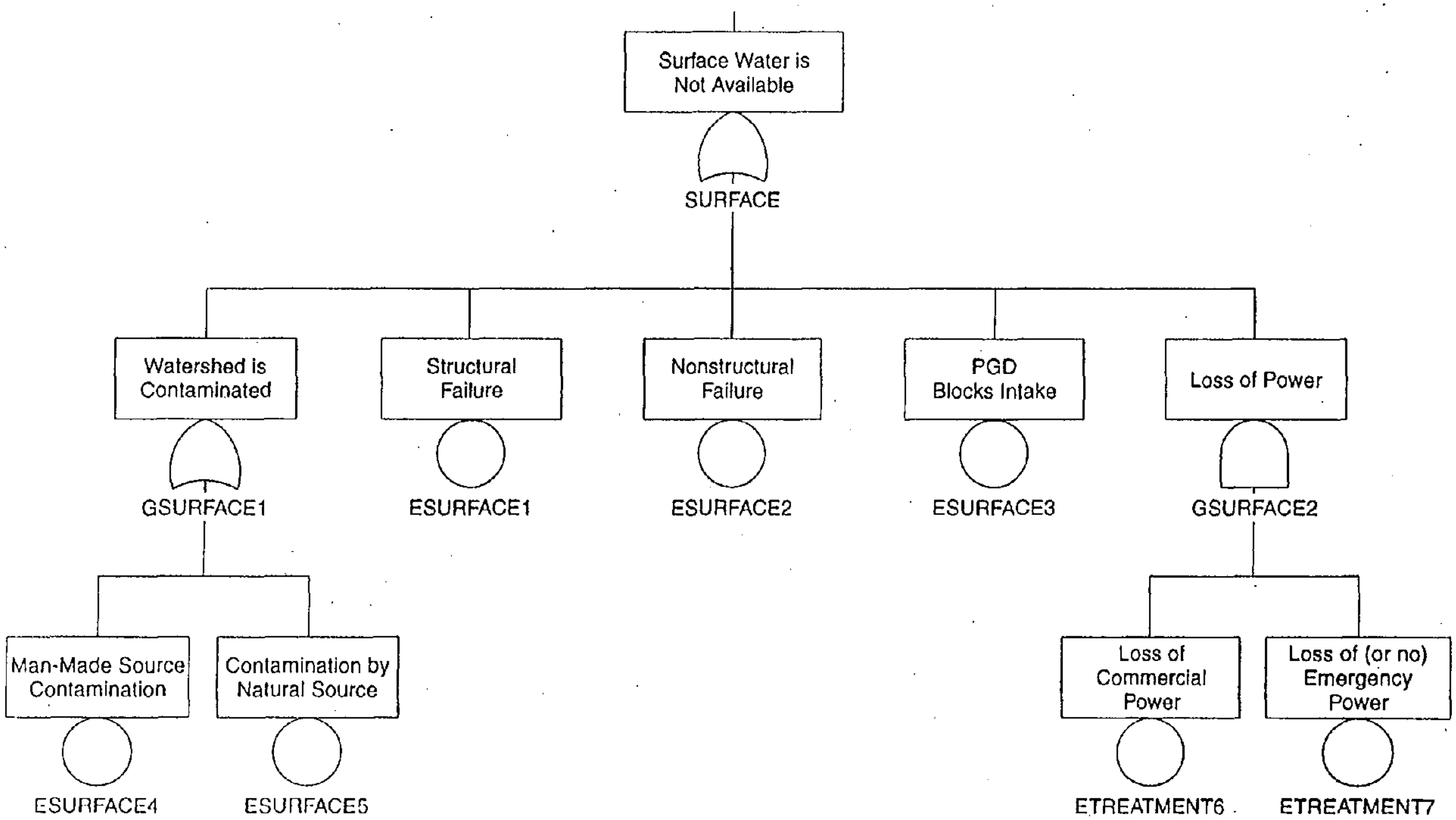

Figure 5-4
Water System Fault Tree (Page 7 of 8)

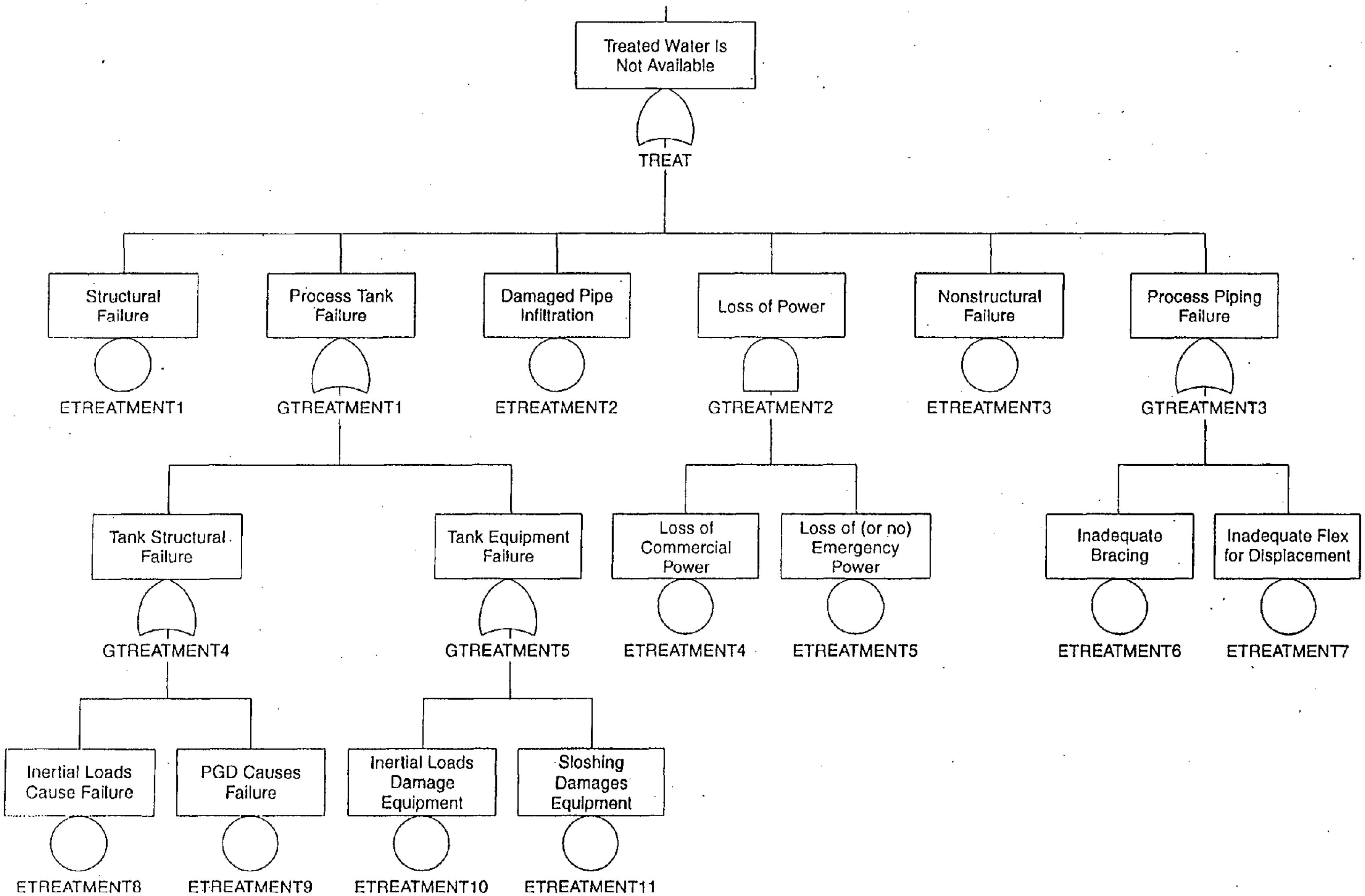

Figure 5-4
Water System Fault Tree (Page 8 of 8)

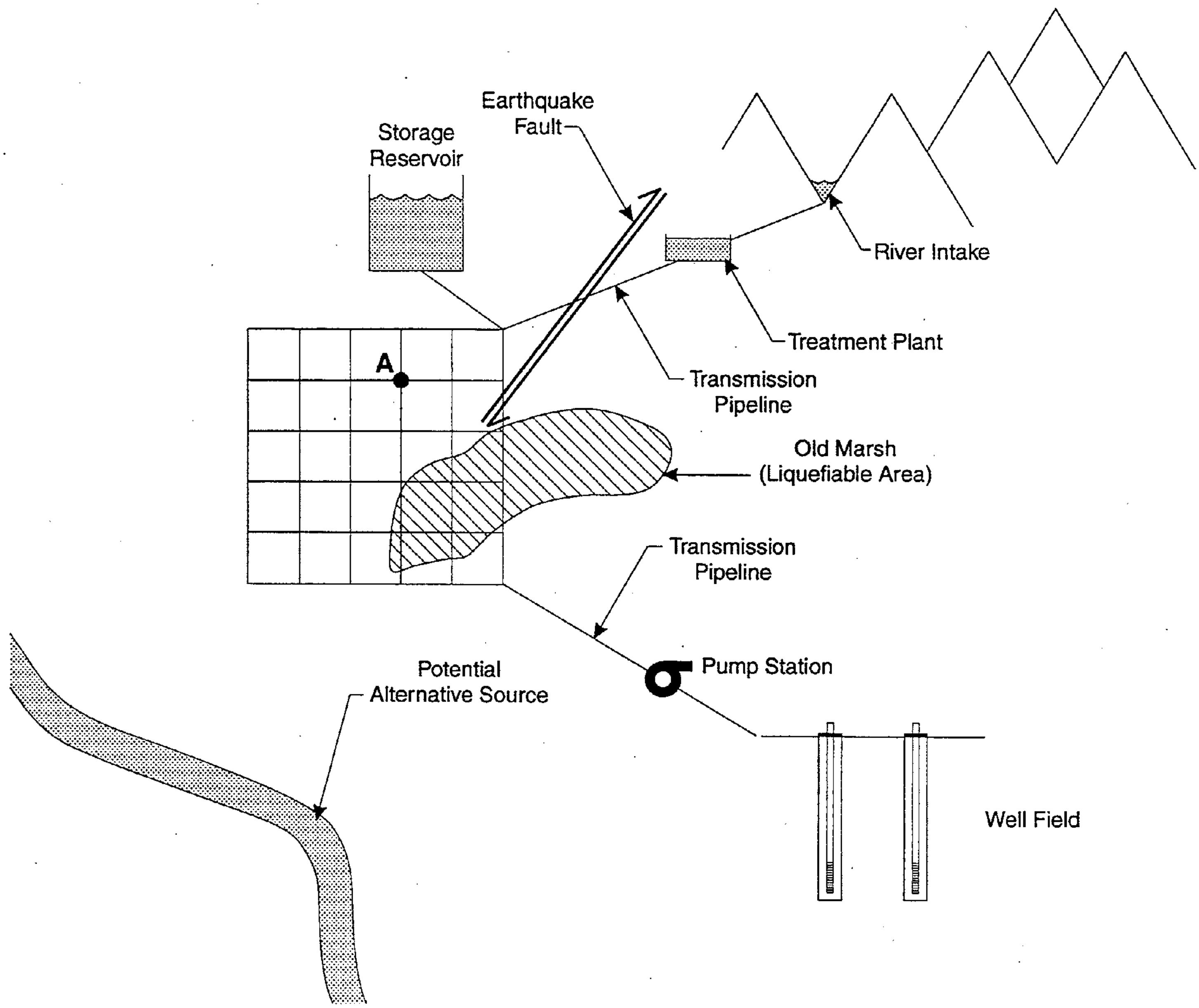

Figure 5-5
Hypothetical Water System Schematic

Event Name	Num. of Occ.	Probability of Failure	Fussell-Vesely Importance	Risk Reduction Ratio	Risk Increase Ratio
ESOURCE1	154	1.000E+000	1.372E-002	1.014E+000	1.000E+000
ESTORAGE1	81	7.500E-001	1.117E-002	1.011E+000	1.000E+000
ESTORAGE2	81	4.500E-001	1.459E-003	1.001E+000	1.000E+000
EPIPES4	1	1.000E+000	7.195E-004	1.001E+000	1.000E+000
EPIPES3	1	8.500E-001	7.195E-004	1.001E+000	1.000E+000
EWELL4	14	4.500E-001	5.183E-004	1.001E+000	1.000E+000
ESOURCE3	22	8.000E-001	3.883E-004	1.000E+000	1.000E+000
EPUMP2	2	7.500E-001	3.111E-004	1.000E+000	1.000E+000
ESURFACE6	22	9.500E-001	3.103E-004	1.000E+000	1.000E+000
ESURFACE7	22	7.500E-001	3.103E-004	1.000E+000	1.000E+000
ESURFACE5	22	6.500E-001	2.627E-004	1.000E+000	1.000E+000
EWELL9	14	2.500E-001	1.602E-004	1.000E+000	1.000E+000
EWELL8	14	9.500E-001	1.602E-004	1.000E+000	1.000E+000
EPUMP4	2	9.500E-001	1.460E-004	1.000E+000	1.000E+000
EPUMP5	2	5.500E-001	1.460E-004	1.000E+000	1.000E+000
EWELL3	14	2.000E-001	1.240E-004	1.000E+000	1.000E+000
ESURFACE2	22	3.500E-001	1.015E-004	1.000E+000	1.000E+000
EWELL2	14	1.500E-001	8.354E-005	1.000E+000	1.000E+000
ESOURCE2	14	1.000E-001	5.026E-005	1.000E+000	1.000E+000
EWELL1	14	1.000E-001	5.026E-005	1.000E+000	1.000E+000
EPUMP1	2	2.000E-001	3.718E-005	1.000E+000	1.000E+000
ESURFACE3	22	1.500E-001	3.564E-005	1.000E+000	1.000E+000
EWELL7	14	5.000E-002	2.278E-005	1.000E+000	1.000E+000
EWELL10	14	5.000E-002	2.278E-005	1.000E+000	1.000E+000
ESURFACE4	22	5.000E-002	1.082E-005	1.000E+000	1.000E+000
EPUMP3	2	5.000E-002	7.984E-006	1.000E+000	1.000E+000
EPIPES2	1	5.000E-002	6.683E-006	1.000E+000	1.000E+000
EWELL6	14	1.000E-002	4.226E-006	1.000E+000	1.000E+000
EWELL5	14	1.000E-002	4.226E-006	1.000E+000	1.000E+000
ESURFACE1	22	1.000E-002	2.087E-006	1.000E+000	1.000E+000
EWATER1	1	1.000E-002	1.283E-006	1.000E+000	1.000E+000
EWELL11	14	1.000E-002	4.149E-008	1.000E+000	1.000E+000
EWELL12	14	1.000E-002	4.149E-008	1.000E+000	1.000E+000

Figure 5-6 Basic Event Relative Importance

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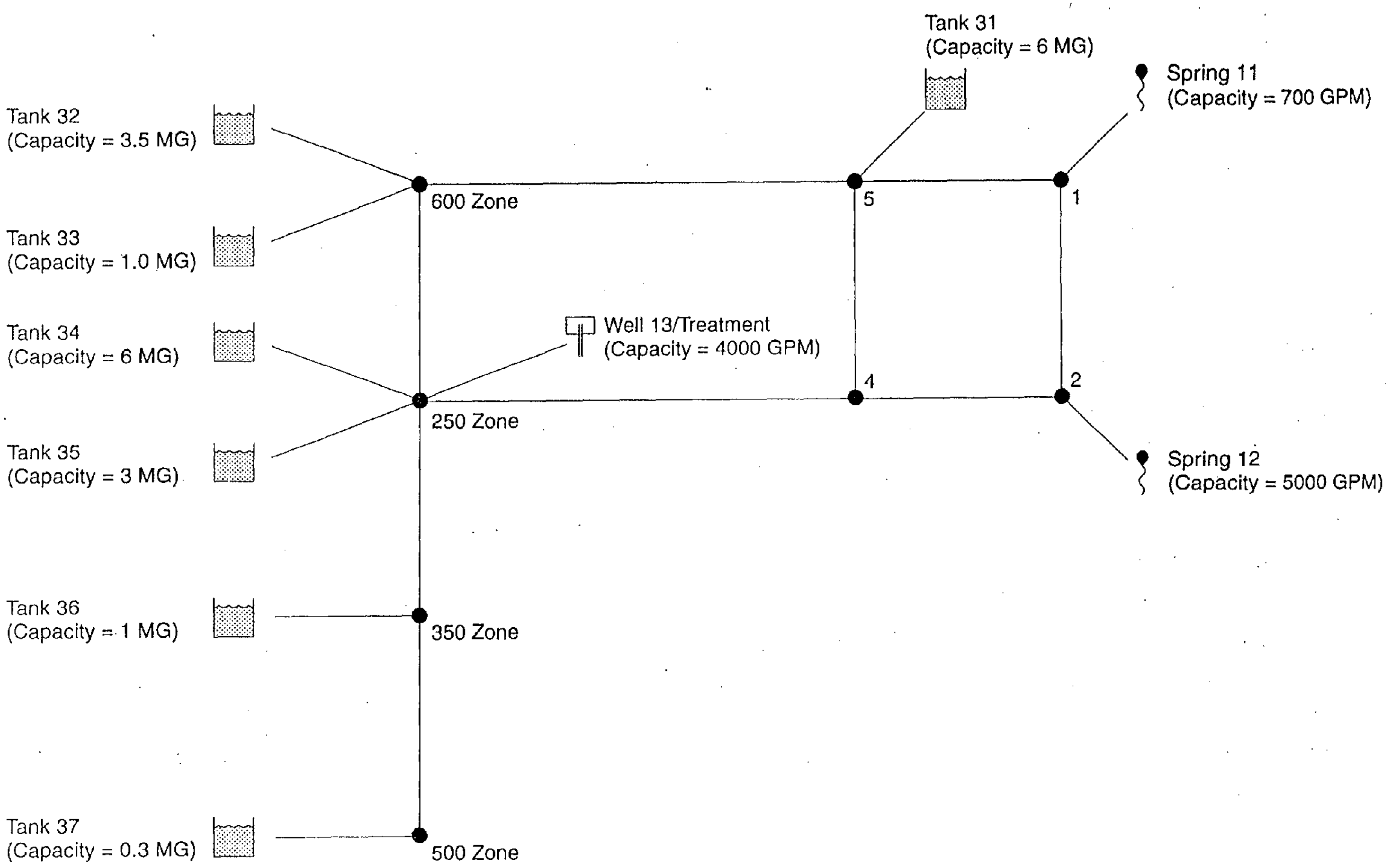

Figure 5-7
Simplified Network Model

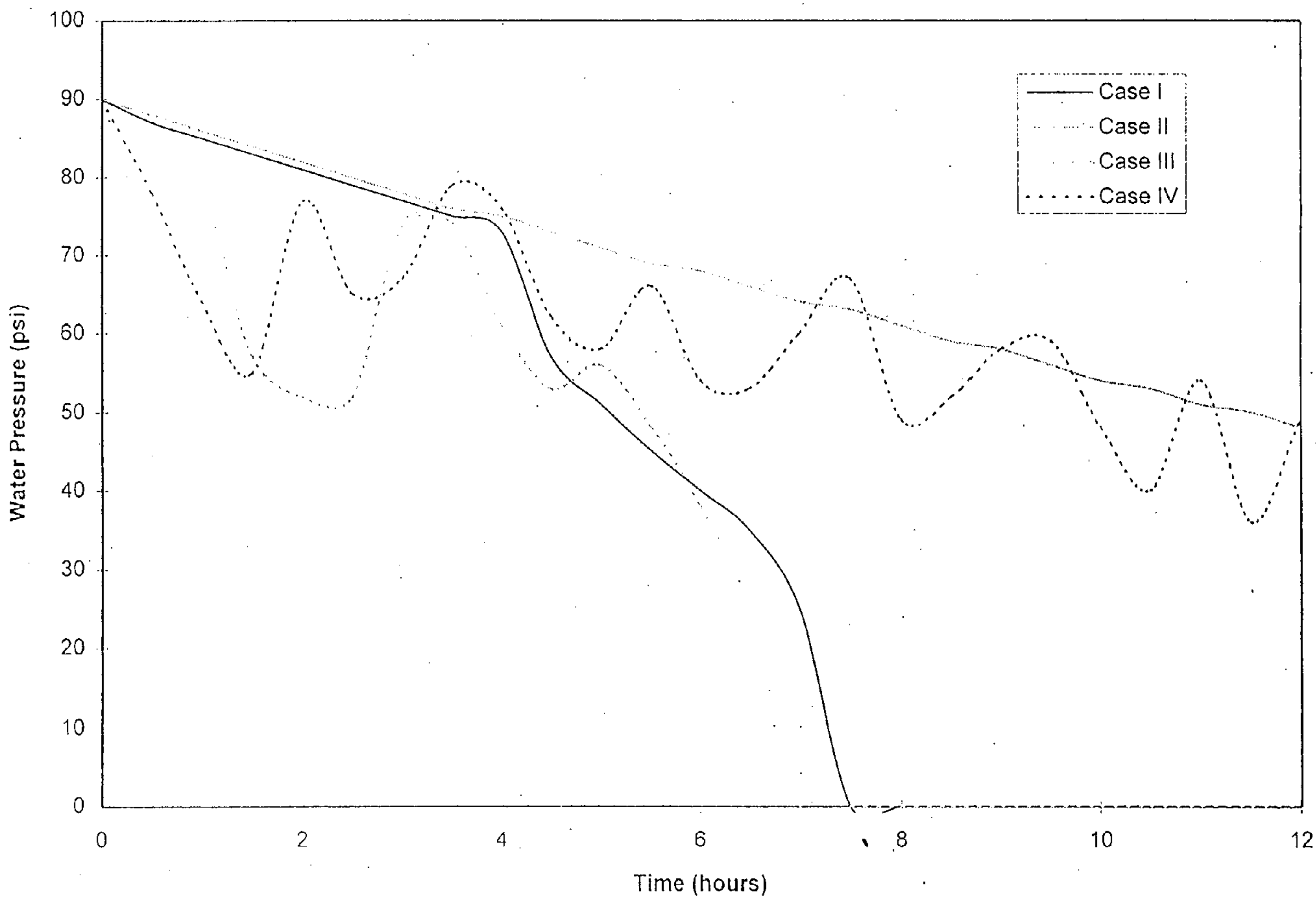

Figure 5-8
600 Zone Post-Earthquake Water Pressure

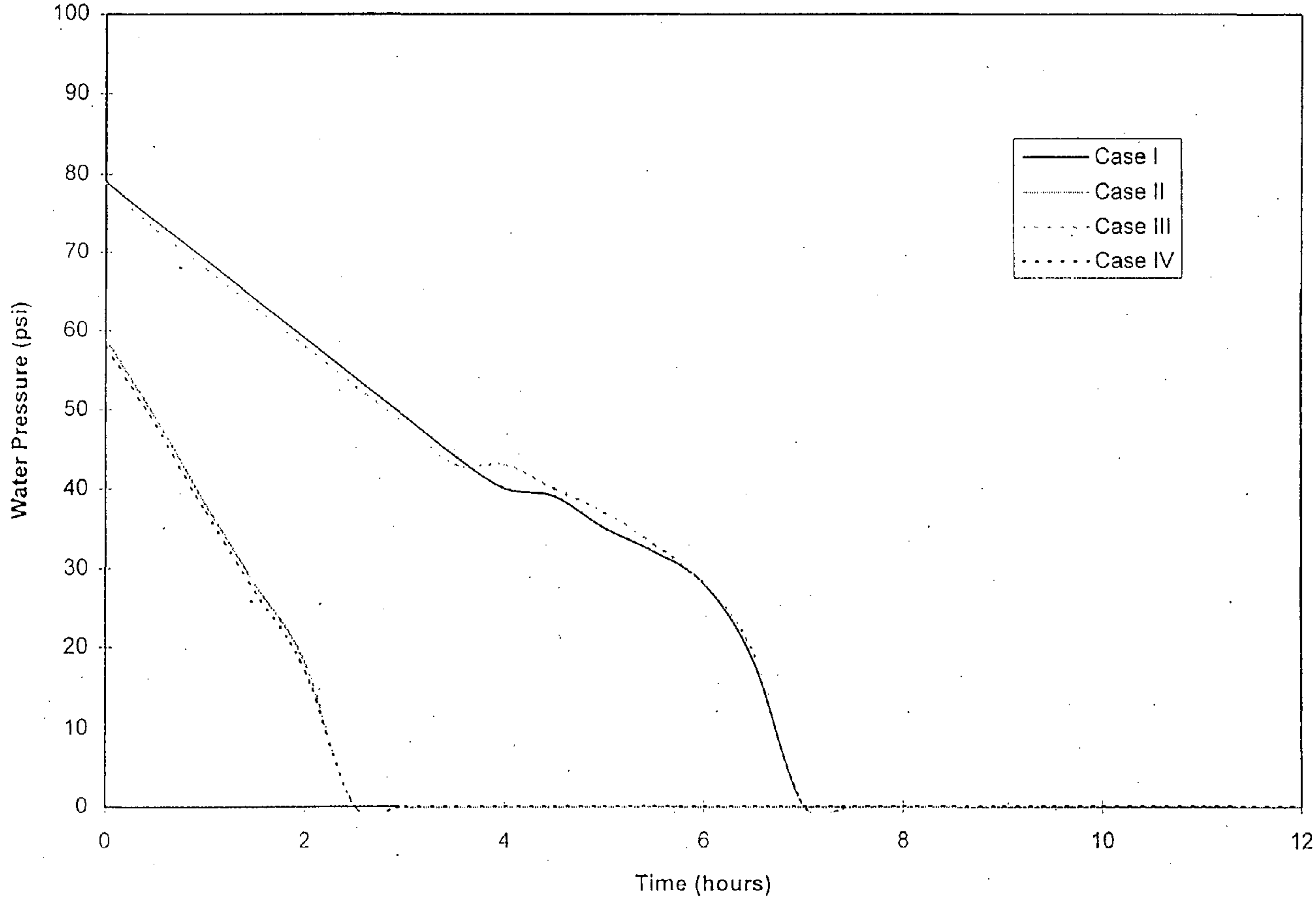

Figure 5-9
250 Zone Post-Earthquake Water Pressure

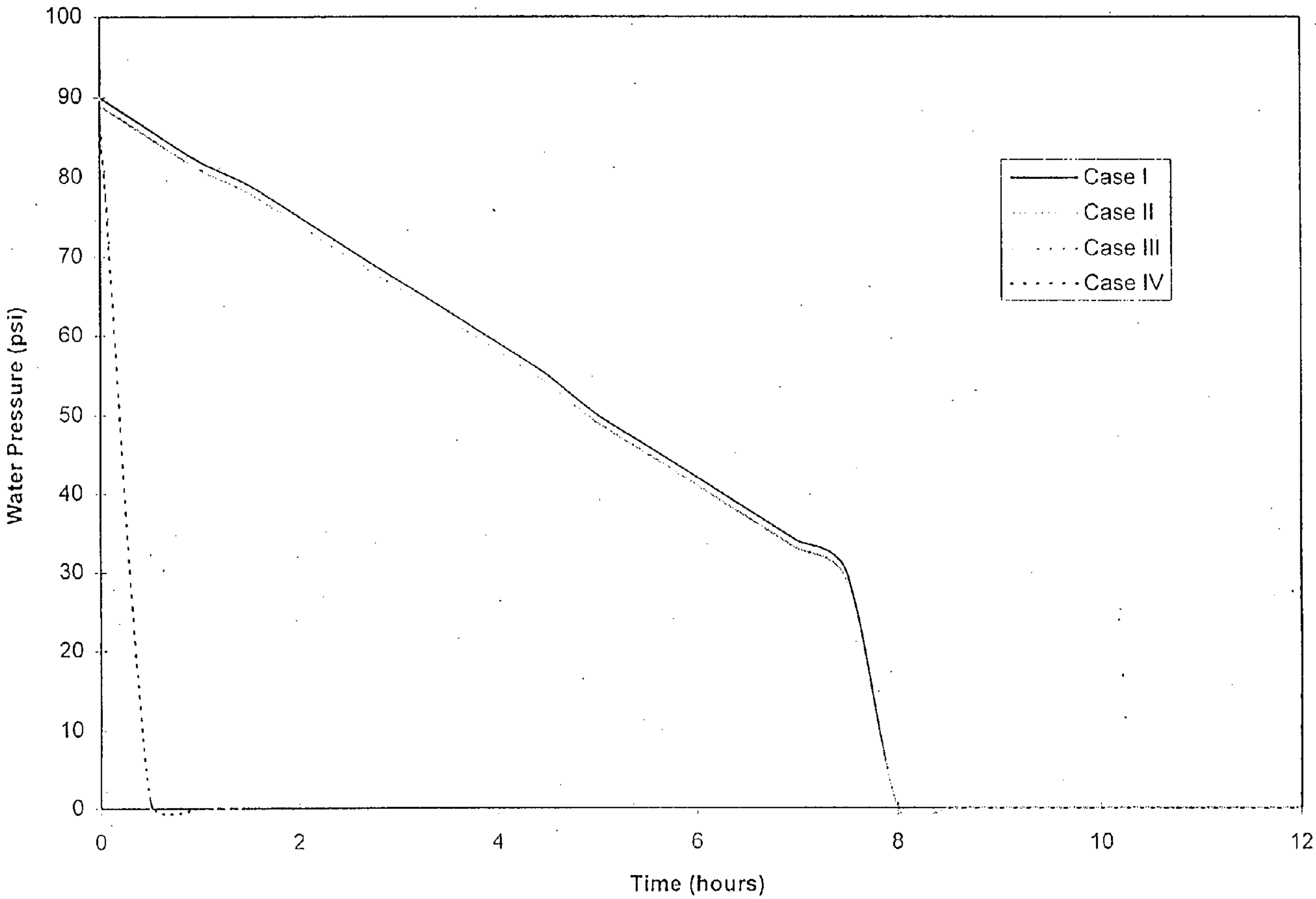

Figure 5-10
350 Zone Post-Earthquake Water Pressure

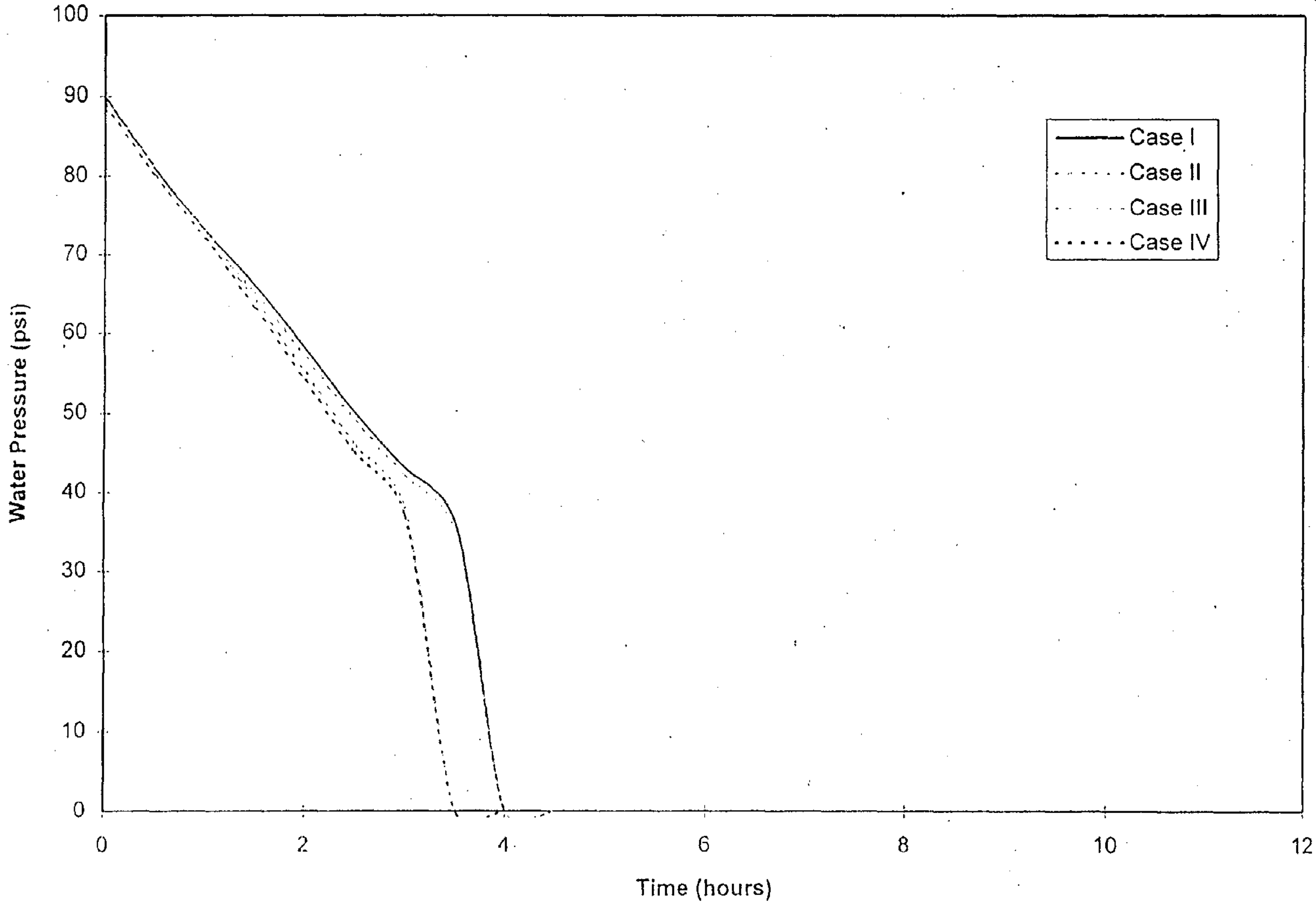

Figure 5-11
550 Zone Post-Earthquake Water Pressure

6.0 SYSTEM COMPONENT DAMAGE

6.1 PIPELINE COMPONENT DAMAGE

Significant damage to buried pipeline is a common occurrence after moderate to large earthquakes. For example, the Los Angeles Department of Water and Power (LADWP) reported over 1,500 repairs to buried pipe occasioned by the 1994 Northridge, California Earthquake (magnitude 6.7). More recently, the 1995 Kobe, Japan Earthquake (magnitude 6.9) resulted in over 1,600 repairs to the water supply distribution system which consisted of roughly 3,900 km of pipe. In Kobe, the pipe damage has been attributed to both wave propagation effects as well as permanent ground deformation (PGD) effects. As expected, the damage rates (repairs per km) were heaviest in soft soil areas and in areas subject to PGD.

This section presents a brief overview of commonly observed seismic damage mechanisms for buried pipelines, as well as a review of both analytical and empirical evaluation techniques. This information is presented for both segmented pipe (e.g., cast iron pipe with bell and spigot joints), as well as continuous pipe (e.g., steel pipe with welded joints).

6.1.1 Damage Mechanisms

For large diameter segmented pipe such as concrete cylinder pipe (e.g., Lock-Joint pipe), seismic damage most frequently occurs at pipeline joints. The two damage mechanisms of interest are joint pull-out (i.e., the bell and spigot ends separate when the joint is subject to axial tension) and joint crushing (i.e., a telescoping failure when the joint is subject to axial compression). For example, most of the seismic damage to concrete cylinder pipe in Mexico City occasioned by the 1985 Michoacan Earthquake (magnitude 8.1) was due to joint crushing.

For a smaller diameter segmented pipe, joint pull-out and crushing are frequently observed, as well as damage in the pipe barrel itself (i.e., round flexural cracks) and cracking at fittings such as Tees and elbows. For example, essentially all of the damage to asbestos cement (AC) water pipe in Limon Costa Rica occasioned by the April 22, 1991 earthquake (magnitude 7.5) was due to pipe barrel breaks. However, for the same event, damage to cast iron and PVC pipe was, more or less, equally distributed between pipe segment breaks (~43%), joint pull-out and crushing damage (~28%), and fitting damage (~29%).

For continuous pipelines, the most commonly observed damage mechanism is local buckling (wrinkling) of the pipe wall. In older lines, specifically those with oxy-acetalene welds, tensile rupture at the welded joints is also common. In fillet welded pipe with slip joints (i.e., the steel pipe equivalent of bell and spigot joints), local buckling or tensile rupture are possible due to additional stresses induced by the eccentricity at the joint.

Beam buckling (i.e., Euler buckling in which the line deforms as a sine curve, breaking through the ground surface) has been observed in some smaller diameter lines, most frequently in oil/gas gathering fields where the depth of cover over the line is small. However, for continuous water lines, which typically have larger diameters and burial depths, beam buckling is not expected. That is, when these lines are in a ground compression area, they wrinkle before experiencing beam buckling.

6.1.2 Analytical Evaluation Techniques

Since there are no consensus-based, national seismic design codes in the U.S. for buried pipelines, there are no "accepted" analysis procedures, limit states or "design" earthquakes. There are, however, limit states and analysis methodologies suggested by various researchers. Currently in seismic retrofit and similar projects, the "design" earthquake (475 year event or maximum credible event or whatever) is typically established by the design team on a case-by-case basis.

In discussing analytical evaluation techniques, it is useful to distinguish between the two types of seismic hazards for buried pipe. There are wave propagation (transient pipe deformation due to traveling seismic wave effects) and PGD, permanent pipe deformation at fault crossings due to fault rupture, landslides, or liquefaction induced lateral spreading. In terms of availability and current usage, analysis methodologies for continuous pipe subject to PGD are more common.

6.1.2.1 Continuous Pipe

For continuous pipe subject to PGD, the hazard is characterized by the amount and spatial extent of the ground movement. The allowable pipe strain for the local buckling limit state is typically established as a fraction of the pipe t/R ratio, where t is the pipe wall thickness and R is the pipe radius. A value in the range of $0.15\ t/R$ to $0.20\ t/R$ is frequently used (Hall & Newmark, 1977). The allowable pipe strain for the tensile rupture limit state is almost always larger, and related to the plastic strain capacity of the pipe material. For example, a value of 3% to 5% has been suggested for use with high strength (x-grade) pipe with modern (i.e., electric-arc) welds. Establishing an appropriate tensile rupture limit state for older, existing, oxy-acetalene welded pipe is more difficult. For these cases, the capacity is directly related to welder workmanship which is quite variable.

For a number of different types of PGD, there are closed form (i.e., hand calculation) analysis procedures which can be used to estimate strain in continuous pipe. Examples include strike-slip faulting which puts the line in tension (Kennedy et al, 1977), and lateral spreading in which the ground movement is parallel to the pipe axis. For these and other cases, it often proves more practical to use a finite element pipe model to determine pipe strain for a given amount of PGD. For both hand or computer calculations, one needs to characterize the load-deformation relation at the soil-pipe interface. In practice, the elasto-plastic soil springs recommended in the ASCE guideline (ASCE, 1984) are often used.

For analysis or design of continuous pipe subject to wave propagation, the hazard would most likely be characterized by the induced ground strain due to traveling wave effects. For low levels of ground strain, there would be relatively little slippage at the soil pipe interface and hence the resulting pipe strain would be only slightly less than the ground strain. For moderate to high levels of ground strain, extensive slippage at the soil pipe interface occurs, and the resulting pipe strain is related to soil friction forces and the wavelength of the seismic excitation. Closed form analysis procedures are available for these cases (O'Rourke & Elhmadi, 1988), and pipe limit states discussed above would likely apply. The presence of fittings (tees, elbows) in continuous pipe subject to either wave propagation or PGD hazards tends to increase the potential for seismic damage. This is due to induced flexural stresses and bending moments at the fittings. Various closed form analytical procedures have been proposed for wave propagation hazard; however, there appears to be little or no agreement on the most appropriate approach.

Another consideration which complicates analytical evaluations for existing steel pipelines is corrosion. Corrosion weakens steel lines by locally reducing the pipe wall thickness. Although post-earthquake observations have shown that corrosion increases pipe damage rates, appropriate limit states and stress concentration factors are not well established.

As indicated above, there appears to be general agreement on some of the key elements (specifically, appropriate limit states and soil springs) needed for analytical evaluation of straight runs of corrosion-free, modern (i.e., electric-arc welded) continuous pipe subject to seismic effects. In contrast, analytical evaluation of segmented pipelines is somewhat more complex and it appears there is less agreement on appropriate procedures.

6.1.2.2 Segmented Pipelines

Analytical evaluation of segmented pipe for seismic effects is complicated by the presence of joints. Models for these types of pipe need to include the joint flexibility (i.e., axial load-deformation and moment-rotation). Also, the presence of segmented joints introduce additional failure modes, specifically joint pull-out and telescopic crushing.

Some information exists on appropriate limit states for segmented pipeline joints. For joint pull-out, limited laboratory tests suggest that bell and spigot joints begin to leak when the relative axial extension at the joint is roughly half of the total joint depth. Estimates of the crushing strength of joints have been based upon strength of material calculations and joint geometry. For transverse PGD (ground movement perpendicular to the pipe longitudinal axis), a limit state based upon manufacturer's recommendations for the maximum angular offset for pipe laying purposes have been suggested.

Analysis procedures exist for straight runs of segmented pipe subject to wave propagation. These models indicate that axial effects (i.e., axial/longitudinal strain in the pipe barrel, and axial/longitudinal extension and contraction at the joints) are more important consideration than transverse effects (i.e., lateral bending/flexure in the pipe barrel, and rotation at the joints). As one might expect, relatively simple models for axial effects show that the pipe segment strain is a decreasing function of the ratio of joint axial stiffness to pipe barrel axial stiffness while the joint displacement is an increasing function of the ratio. Practical application of these results is limited by two problems. First, realistic values for joint stiffeners are available for only a limited number of materials. Secondly, the simple models only provide estimates for the average response (e.g., axial extension at an average joint). However, since even large wave propagation excitation results in damage to only a small fraction of all joints (e.g., 1 in 500), failure in these segmented systems is governed by behavior in the tail of the probability distribution, as opposed to the behavior of the average joint.

Computer-based (Monte Carlo) analysis methodologies have been developed which incorporate the variability of joint characteristics (e.g., joint axial stiffness) from joint to joint (Elhmadi & O'Rourke, 1990). Although these methodologies provide estimated damage ratios (repairs per kilometer of pipe) within a factor of 2 to 5 of observed values, they require as input a measure of the variability of joint characteristics (i.e., probability density function for joint stiffness). The availability of such information is very limited.

In terms of the PGD hazard, there are some available analysis procedures. For example, computer-based models have been used to evaluate segmented pipe crossing strike-slip faults. Also simplified closed form methodologies have been developed for segment pipe subject to transverse PGD (ground movement perpendicular to the pipe's longitudinal axis).

However, existing analysis/design methodologies for segmented pipe subject to either wave propagation or PGD are infrequently used in practice. More frequently, seismic design of segmented pipe simply involves the installation of special seismically resistant joints at particularly vulnerable locations (e.g., areas subject to PGD or isolated areas where the subsurface conditions change abruptly).

6.1.3 Empirical Evaluation Techniques

Often the first step in the seismic upgrade of a buried pipeline system is an evaluation of the likely amounts of damage in the existing system due to potential earthquakes. For buried pipe components, empirical correlations between observed seismic damage and some measure of ground motion are typically used. These empirical relations are frequently used in practice.

The ground motion has been characterized by the peak ground acceleration, the peak ground velocity, the amount of PGD movement or by the more general Modified Mercalli Intensity (MMI). In some of the empirical relations, damage due to both the wave propagation hazard and the PGD hazard are combined, while others address damage due to only one of the hazards. The following sections briefly summarize the existing relations.

6.1.3.1 Combined Damage

Katayama et al, (1975) developed one of the first relations, primarily for segmented cast iron pipe, in which the damage rate (repairs per unit lengths) due to both wave propagation and PGD is plotted as a function of peak ground acceleration. Other relations, again primarily for cast iron pipe, have been developed wherein combined damage is plotted versus MMI. In one of these relations, the influence of diameter (less damage for larger diameters) and soil type are included.

6.1.3.2 Wave Propagation Damage

It appears that Eguchi (1983) was the first to develop separate empirical relations for wave propagation (ground shaking) damage. In these relations, the ground motion is characterized by MMI. The most recent version of the Eguchi relation for ten different pipe materials is shown in Figure 6-1.

Other researchers have developed different empirical relations for wave propagation damage as functions of the peak ground velocity. In one, a single curve is presented for common brittle water pipe materials (CI, AC, and CONC), while in others separate curves are available for various combinations of pipe material, joint type, soil corrosiveness, and diameter.

6.1.3.3. PGD Damage

Eguchi (1983) also developed empirical vulnerability relations for buried pipe subject to fault rupture, landslides and liquefaction (lurching). For the fault rupture relation, damage is a function of the amount of fault offset. A more recent relation for non-fault rupture PGD has been developed (Harding Lawson et al, 1991). The bilinear curve shown in Figure 6-2 is based upon cast iron pipe damage data for the 1906 San Francisco and the 1989 Loma Prieta events. Engineering judgment was used to extend the relation to other pipe materials. In these curves the damage ratio is a function of the amount of ground movement. This relation is used in HAZUS, a FEMA-developed earthquake loss estimation methodology (RMS, 1996).

Other empirical PGD damage relations have also been developed. In one, the damage rate is primarily a function of ground movement (as in Fig. 6-2), but also includes factors related to pipe material and joint type. For example, asbestos cement and cast iron pipe with rubber gasketed joints are expected to have about 25% less damage than the same material with cemented joints. While in another relation, the expected damage to cast iron pipe with rigid joints is roughly a factor of four larger than that for modern welded steel pipe.

6.1.3.4. System Performance

Knowing the expected damage in a pipeline network, the direct loss (i.e., expected repair costs) can be calculated. However, in terms of impact upon the population served by the system and particularly the ability to fight post-earthquake fires, system functionality is an equally important measure. Such functionality estimates may be developed from a hydraulic model of the damaged system. In such models, one needs to distinguish between leaks and breaks. That is, a pipeline break has a much larger impact on functionality than a pipeline leak. One study in the Puget Sound, Washington area (Ballantyne et al, 1990) indicated roughly 85% of the repairs were leaks, while the remaining were breaks. In the HAZUS methodology, wave propagation damage is assumed to result in 80% leaks and 20% breaks, while for PGD damage is assumed to result in 20% leaks and 80% breaks.

Figure 6-3 shows the results of a number of functionality studies, in which the serviceability index is plotted as a function of the average number of breaks per unit length. The curve labeled NIBS is currently used in the HAZUS methodology (RMS, 1996).

6.2 TANKS

6.2.1 Earthquake Response of Flat-Bottomed Tanks

Flat-bottomed vertical liquid storage tanks have sometimes failed with loss of contents during strong earthquake shaking. In some instances, the failure of storage tanks has caused disastrous consequences. Some examples include: (1) failure of numerous water storage tanks, both above and below ground, in the 1971 San Fernando, California, the 1980 Livermore, California, the 1985 Chilean, the 1991 Costa Rica, the 1992 Landers, California, and the 1994 Northridge, California earthquakes; (2) fires causing extensive damage to oil refineries in the 1964 Niigata, Japan earthquake and the 1991 Costa Rica earthquake; (3) polluted waterways in the 1978 Sendai, Japan earthquake; and (4) fires and failure of numerous oil storage tanks in the 1964 Prince William Sound, Alaska earthquake (Summers and Hults, 1994).

The response of unanchored tanks during earthquakes is highly nonlinear and much more complex than implied in available design standards. Seismic ground shaking generates an overturning force on the tank, which may cause a portion of the tank to lift up from the foundation. The weight of the fluid resting on the uplifted portion of the tank bottom, together with the weight of the tank shell and roof, provide the restraining moment against further uplift. While uplift, in and of itself, may not cause serious damage, it can be accompanied by large deformations and major changes in the tank wall stresses. Tank uplift during earthquakes has been observed many times, but the amount of uplift has only been recorded on some occasions. Some observations include: (1) a 100 ft diameter, 30 ft high tank uplifting by 14 inches during the 1971 San Fernando, California earthquake; (2) tanks uplifting by 6 to 8 inches during the 1989 Loma Prieta, California earthquake; and (3) a tank uplifting by 18 inches in the 1964 Alaska earthquake.

In general, tanks, especially unanchored tanks, are particularly susceptible to damage during earthquakes. Tank damage during earthquakes usually falls into one of the following categories (Dowling and Summers, 1993):

- Buckling of the tank wall, known as "elephant foot" buckling. This occurs because of high vertical compressive stresses in the portion of the tank wall remaining in contact with the ground (i.e., diametrically opposite the uplifted portion) and as the uplifted portion impacts the ground. The compressive stresses are accompanied by bending stresses introduced into the shell wall because the baseplate prevents the radial deformation which would normally occur under internal pressure. This further increases the tendency to buckle. In many instances, however, buckled tank walls have continued to retain the stored product.

- Breakage of attached inlet/outlet or drain piping due to tank uplift and inadequate piping flexibility. This is one of the most prevalent causes of product loss from storage tanks during earthquakes. Failures can also occur due to relative movement between two different tanks connected by rigid piping.

- Tearing of the tank wall at locations where there are structural discontinuities, such as plate doubling at manholes and pipe nozzles.

- Tearing of tank wall due to overconstrained stairways anchored at the foundation and tank shell.

- Tearing of tank wall due to overconstrained walkways connecting two tanks experiencing differential movement.

- Damage to the tank's roof, possibly followed by spillage of fluid over the tank walls. This can occur if insufficient freeboard is provided to accommodate sloshing of the surface of the tank contents. This type of damage is usually considered only minor but may be important. Roof-supporting columns are also subject to damage from sloshing water. The sloshing water imparts lateral loading on columns and can impart vertical upthrust loads on tank roofs, particularly around the periphery, if sufficient freeboard is not provided. In addition, heavy concrete roofs are susceptible to damage if the structure is not designed to transfer loading to the tank walls and foundation.

- Geotechnical and foundation failures. Differential settlement is one concern, particularly when a tank is founded partially on undisturbed soil, and partially on fill. Landslides, either below or under the tank and above or into the tank, can also be a concern.

- Liquefaction may be a concern if the site is susceptible to liquefaction. This is uncommon because water storage tanks and reservoirs are often located on high ground where liquefaction susceptibility is usually low. In-ground reservoirs constructed with earthen berms may be susceptible to liquefaction particularly if water is leaking from the reservoir, maintaining a saturated soil condition.

- Wire-wrapped and post-tensioned concrete tanks are vulnerable to earthquakes if reinforcing has corroded, or if the roof/wall or wall/bottom joints are not designed to carry earthquake loads. Concrete tank wire wrapping has shown tendencies to corrode in 1960s vintage tanks resulting in tank failure (Ballantyne, 1994). Indications of tank deterioration are vertical cracking, spalling, or staining from tank leakage through the shotcrete. These problems have been mitigated first by stopping the leaks with tank linings followed by re-wrapping the tank with wire or steel bands.

- For concrete tanks, the joint between the tank wall and bottom must be designed to transfer earthquake shear loads. In modern designs, earthquake cables are used between the wall and bottom. These cables allow the wall to move to accommodate strains induced from tank filling, but limit movement in an earthquake. Tanks designed prior to the 1970s did not use earthquake cables (Ballantyne, 1994), and may fail at the wall/bottom connection in an earthquake. For partially buried tanks, the passive earth pressure should prevent shear failures. One solution is to provide a curb around the periphery to limit sliding.

Anchorage of steel tanks is a mitigation alternative for many of the above phenomena. Anchored tanks generally perform well during earthquakes, provided anchorage details are designed in a ductile manner to transfer forces from the shell to the foundation, and the weight of the foundation is sufficient to overcome the overturning moment imposed by the seismic event. The American Water Works Association (AWWA) permits anchorage of steel tanks, although anchored tanks are not generally used for large water storage reservoirs. Anchorage, as a mitigation alternative, is usually only implemented for taller, more slender tanks with higher aspect ratios.

6.2.2 Proposed Methodology for Seismic Evaluation of Tanks

The steps involved in undertaking a seismic hazard mitigation program for existing tanks include:

a. Walkthrough inspection to assess piping, stairway and walkway attachments, and other potential hazards.

b. Analytical assessment of tanks to evaluate the potential for overturning and shell buckling or overstress.

c. Mitigation of seismic hazard. The most commonly used hazard mitigation measures include addition of flexibility to rigid attachments, reduction of safe operating height and, as a last resort, anchorage of the tank or replacement/stiffening of bottom shell courses.

These items are discussed in more detail below.

6.2.3 Walkthrough Inspection

The purpose of the walkthrough is to identify a variety of seismically vulnerable details.

The principal feature which distinguishes the seismic response of unanchored tanks from that of anchored tanks is the large uplift displacements commonly observed around the edge of unanchored tanks. As described earlier, this uplift may induce large tension or compression forces and bending moments in the tank wall, baseplate, and at the intersection of the two. Such forces may lead to severe damage or failure of the tank. Furthermore, the adverse effects of excessive tank uplift can be greatly exacerbated by a variety of commonly encountered tank details. Walkthrough inspection of individual tanks or tank farms should focus on the identification of such details. In many cases, while the tank itself may be found to be structurally adequate, retrofit of a number of these seismically vulnerable details may be deemed necessary. The following material, which describes the walkthrough process, is taken from Dowling and Summers (1993) and Summers and Hult (1994).

The most frequently encountered hazardous details are listed below, together with appropriate retrofit recommendations, and illustrated in Figure 6-4.

a. A common failure mode in tanks has been breakage of piping connected to a tank as a result of relative movement between the tank and the nearest pipe support. Alternatively, if the piping is stronger than the tank wall or baseplate to which it is connected, tearing of the wall or baseplate may result. Piping should not pass directly, with little or no flexibility, from the tank shell or tank bottom to the ground or to rigid concrete walls, basins, pumps rigidly fixed to the ground, etc. Failures of the type described above are typically caused by the details shown in Figures 6-4 (a) through 6-4 (d). In the first three cases, additional piping flexibility should be provided by adding horizontal or vertical bends, or by installing a length of flexible piping. In the fourth case, piping should be rerouted towards the center of the tank or, if the piping is flexible enough, the concrete basin may be extended beyond the pipe/tank connection.

b. Similar failures have also occurred due to relative movement between two tanks connected by a rigid pipe, as shown in Figure 6-4 (e). Again, additional piping flexibility should be provided as described above.

c. Partial loss of contents may result from the type of detail shown in Figure 6-4 (f), where a vertical pipe is rigidly connected to the ground or foundation and also supported rigidly along the wall of the tank. A detail offering a lesser level of risk, but present in many cases, is a tank wall support that consists of a large U-bolt which might appear to be capable of sliding up and down the pipe as the tank lifts and falls. However, it is possible that the U-bolt will "bind" with the pipe due to friction, thereby also forming an essentially rigid connection, and leading to tearing of the tank wall. Any connection along the tank shell judged to be rigid should be replaced by a connection near the shell/roof intersection, coupled with sliding connections or "guides" along the shell wall. In many cases, simply loosening the U-bolts will suffice.

d. Roof access is sometimes facilitated by walkways spanning between the tanks. Typical walkway arrangements are shown in Figure 6-4 (g). In both cases, relative movement between tanks may lead to rupture or tearing of the tank wall or roof. However, whereas the lower walkway arrangement shown in Figure 6-4 (g) may lead to partial loss of tank contents, the upper walkway arrangement will, at worst, lead to damage to the walkway itself and/or to the roof; hence, no loss of contents will result. The distinction between the two arrangements is important since the lower walkway represents a concern that could result in release of water, whereas failure of the upper walkway will likely only result in a level of economic loss. In either case, the required retrofit would take the form of increased walkway flexibility.

   Elevated walkways also represent falling hazards for tanks located with adjacent pedestrian traffic or sensitive equipment. Where necessary, walkways should be attached to the tanks by cables as a secondary means of support to prevent them from falling.

e. Stairways should not be attached to both the tank shell and the foundation; see Figure 6-4 (h). However, only in thin-shelled tanks is such a detail likely to lead to failure of the tank wall and to loss of contents. Again, therefore, a distinction should be drawn between this case and the case of a thick-shelled tank, where such a detail would only result in damage to the stairway itself. In either case, the hazard may be eliminated by attaching the stairway to the tank shell or by eliminating the connection which prevents the stairway from displacing vertically.

Tank piping is usually concentrated in one area, often with an access walkway spanning over the piping, as shown in Figure 6-4 (i). If the walkway is rigidly attached to the ground and if insufficient clearance is provided between the piping and walkway, then tank uplift may lead to impact between the piping and the walkway, resulting in damage to one or the other. However, only with small diameter pipes or thin-shelled tanks is loss of tank contents likely to occur. Otherwise, damage is likely to be confined to the walkway itself. In both cases, the hazard may be mitigated by increasing the piping flexibility, attaching the walkway exclusively to the tank shell, or providing more piping clearance. Another potentially seismically vulnerable detail is the case of a walkway attached to both the tank shell and foundation, as in Item (e) above for stairways.

Tanks anchored with anchor bolts having poor connection details may tear the bottom plate or tank shell resulting in a loss of product. Poor details include anchors which are clipped to the bottom plate, chairs which are unusually short so as not to permit adequate transfer of forces in the bolt to the tank shell, poorly designed eccentric connections, or any detail which will result in the tearing of the tank shell before the anchor bolt yields. This hazard can be mitigated by replacing the connection with one that will exhibit more ductility.

When performing walkthrough inspections, the question as to "how much flexibility is sufficient" must be answered. The assumed value of tank uplift is critical to answering this question. Values of 6 to 8 inches have commonly occurred in the past. Using a value of on the order of 6 to 12 inches of vertical displacement, preferably closer to 12 inches, and on the order of 4 to 8 inches of horizontal displacement (at least in zones of highest seismicity) would be considered prudent and conservative. Actual expected values are a function of tank size, fill height, aspect ratio, and local seismicity and soil type.

6.2.4 Analytical Evaluation

The seismic design methodology for welded steel storage tanks presented in both AWWA D-100 (AWWA, 1985) and API 650, Appendix E (API, 1993) is based on the simplified procedure developed by Housner (1977). Details of the development of the methodology are described in Wozniak and Mitchell (1978). The procedure considers the overturning moment on the tank to be the sum of:

a. The overturning moment due the tank shell and roof, together with a portion of the contents which moves in unison with the shell, acted on by a level of horizontal acceleration. This is termed the impulsive component.

b. The overturning moment due to that portion of the tank contents which moves in the first sloshing mode (i.e., independently of the tank shell) acted on by a level of horizontal acceleration. This is termed the convective component.

Resistance to the overturning moment is provided by the weight of the tank shell and roof and by the weight of a portion of the tank contents adjacent to the shell. The structural adequacy of the tank is determined by a "stability ratio," which is a measure of the ratio of the overturning moment to the resisting moment. The value of the stability ratio must not exceed 1.54 (AWWA, 1985).

Extensive experimental studies and observations during past earthquakes have demonstrated that the radial length of the uplifted bottom plate, and hence, the actual liquid weight resistance which is mobilized during an earthquake is underestimated by the AWWA uplift model. The reasons for this are that the AWWA model does not account either for the in-plane (membrane) stress in the bottom plate, or for the dynamic nature of the tank response. The model also calculates a somewhat narrow compression zone at the toe of the tank, thus leading to large compressive stresses in the tank shell for relatively low overturning moments. Finally, the AWWA approach does not account for the effect of foundation flexibility on the tank wall axial membrane stress distribution.

Although the AWWA methodology is known to be somewhat conservative and the condemning existing tanks for failing to meet the AWWA criteria may not be deemed appropriate, the criteria are the basis of the current seismic design practice and serve as a good benchmark. Large exceedance of specific provisions should be taken as an indication that retrofits may be necessary.

There are several alternatives to the AWWA methodology that might be considered for use in evaluation of existing tanks. One such method is a modified version (Dowling and Summers, 1993; Summers and Hults, 1994; and ASCE Task Committee on Seismic Evaluation and Design of Petrochemical Facilities, 1997) of a method developed by Manos (1986) presented herein. Manos' method is based on experimental studies, as well as on observed behavior of unanchored tanks during past earthquakes. Instead of trying to model the complex uplifting plate behavior, Manos assumes a stress distribution at which the shell will buckle and solves for the resisting moment produced by the sum of the stresses. This resisting moment can then be compared to the overturning moment and the resisting acceleration solved for.

The method proposed herein for evaluation of unanchored storage tanks is based on that of Manos, but includes some important variations. The most notable of these are (Dowling and Summers, 1993):

a. Tank anchorage is recommended in zones of high seismicity whenever the ratio of safe operating height to tank diameter exceeds two. Based on the data presented in Manos, and the higher level of risk for taller tanks, this is believed to be the upper limit of applicability of the Manos method.

b. The allowable compressive stress in the tank shell should not exceed 75% of the theoretical buckling stress, as presented in Manos, nor should it exceed the material yield strength. This last requirement is significant for thicker-walled tanks. Note that an increase in the allowable compressive stress beyond 75% of the theoretical buckling stress may be justified under certain circumstances.

c. The compressive force in the tank shell should not exceed the total weight of the fluid contents. This has the effect of imposing an upper bound on the resisting moment.

6.2.5 Validation of Tank Assessment Methodology

The most widely used codes for the seismic design and evaluation of steel water tanks in the United States are the AWWA D-100 for welded steel tanks and the AWWA D-103 for bolted steel tanks. The methodology presented in these standards is believed to be based mostly on analytical work, with limited validation from actual performance of tanks during earthquakes. Manos (1986) developed an alternate methodology for the seismic evaluation of existing tanks. His method is based primarily on experimental studies and observed performance of unanchored tanks during past earthquakes. Since then, a significant amount of tank performance data from several recent earthquakes has become available. It is considered timely that these data be evaluated and analyzed to test the applicability of the AWWA and Manos methodologies.

Dowling and Summers (1993) have argued that for certain cases, the AWWA methodology might be considered conservative in predicting tank damage during earthquakes, and hence, while this method may be appropriate for design of new tanks, the alternate methodology (Manos) may be more appropriate for evaluating existing tanks. The basis for their argument is through comparison with the method proposed by Manos, which is based in part on observed tank performance.

To realistically assess either of the two approaches, damage data from past earthquakes should be correlated with the predictions made by each approach. Results from such analyses can be relied upon only if complete tank attribute and performance data are available in addition to knowledge of the actual ground accelerations to which the tanks were subjected. The approach undertaken in this study was to first compile all readily available data on 85 tanks during the following four earthquakes: the 1985 Costa Rica, 1989 Loma Prieta, the 1992 Landers, and the 1994 Northridge. Tables 6-1 through 6-4 show the data for each of the tanks. It is clear from the tables that the data for most of the tanks are incomplete with either one or multiple attributes missing. However, considering this is the first attempt to compile these data, the results are still encouraging, because most of the missing data could probably be collected with limited further effort. However, such effort was beyond the scope of the present study.

A review of Tables 61- through 6-4 show that a number of the tanks have data which are difficult to collect years after the earthquake. The missing data mostly consist of either the thickness of the tank shell and/or bottom plate or estimates of ground acceleration at the site. Accurate shell thickness data can be obtained either by locating as-built drawings of the tanks or by ultrasonic measurements. The nondestructive nature of the ultrasonic measurements make them relatively easy to conduct; however, the difficulty lies in accessing some of the tank sites (owner permission is needed) for which these data are missing and, of course, in assuming the as-measured thicknesses (today) will be equivalent to those at the time of the earthquake.

Estimates of actual ground motion at many of the tank sites have also been difficult to obtain at this stage for two reasons. One, the precise location of some of the tanks was for this stage unknown and, therefore, the acceleration at the site could not be estimated. This location information, however, is relatively easy to obtain with a little extra effort. The second reason is more complicated. Very few of the tank sites have an onsite accelerometer or one located close enough to make a confident prediction of the higher frequency accelerations at the site, which generate the impulsive forces in the tanks. In most cases, these forces are the primary contributor to the large compressive stresses in the tank shell which have caused shell buckling commonly observed during earthquakes. The ground motions felt by the tanks can only be predicted using either empirical attenuation relationships or acceleration contours estimated for the earthquake based on regionally recorded ground motions. Both these methods have large uncertainties associated with them, and they also do not accurately consider local topographic or subsurface amplification effects at the tank site. Estimated accelerations at the tank site are therefore approximate and could easily differ from the actual accelerations by ± 50%.

During this project, the main effort was spent in compiling as much data as possible. Out of the 85 tanks for which data were obtained, only 11 tanks had enough data to conduct an analysis using the AWWA and Manos methodologies. Six tanks were from the Landers earthquake and five were from the Northridge earthquake.

The following subsections describe the data collected and some preliminary observations made from the results of the demonstration analyses.

6.2.5.1 Ground Motions

Ground motions were estimated at known tank-site locations. These locations were known for 14 of the tanks for which seismic performance data from the Landers Earthquake were available. However, of the 45 tanks for which seismic performance data for the Northridge Earthquake were available, the precise locations of only 23 of them were known. Four of the tanks located in Barstow during the Landers Earthquake were close to an accelerograph station and the recording from this station was considered a reasonable approximation of the actual ground-motion excitation to the tanks. Accelerations for the remaining 10 tank sites in the Landers area were estimated using the median attenuation relationship of

Abrahamson and Silva (1997). Out of these 45 tank sites for the Northridge earthquake, only five sites were sufficiently close to accelerograph stations to confidently estimate actual accelerations that the tanks may have experienced. For each of the remaining tank sites, median estimates of peak ground acceleration (PGA) were made using the empirical attenuation relationship by Abrahamson and Somerville (1996), which was derived from the accelerogram data recorded during the Northridge Earthquake.

The site-dependent AWWA method and the Manos method require estimates of the 2 percent damped spectral acceleration (for the first impulsive mode) and 0.5 percent damped spectral acceleration at the period of the first sloshing mode (for convective response) rather than peak ground acceleration. The approaches to estimate the spectral acceleration at each of the tank sites were the same as those for estimating peak ground acceleration.

6.2.5.2 Tank Physical Data

Tank attribute data necessary to perform this analysis include the diameter and height of the tank, fill height of liquid at the time of the earthquake, and thicknesses of bottom shell course and bottom plate. Other information such as roof plate thickness and subsurface conditions is desirable but less essential. Diameter and height data were available for all of the 85 tanks. With the exception of 18 tanks from the Northridge Earthquake, fill height data at the time of the earthquake were either available or could reasonably be estimated. These estimates were based on assumptions of the tanks being full based on evidence of roof damage due to sloshing, or on the presumption that the tanks were full during the early morning hours when the Northridge and Landers Earthquakes occurred.

Thickness data for either the bottom shell course or the bottom plate or both were missing for the majority of the tanks. Out of a total of 85 tanks, both the bottom shell and bottom plate thicknesses data were available for only ten tanks. Only bottom plate thickness data were missing for 14 of the remaining 75 tanks. Typically, the bottom plate thickness of most small to medium size tanks is 0.25 inches or equal to the thickness of the bottom shell course if the tanks have not been constructed with a sketch plate. If this assumption is made, then the number of tanks for which analysis could have been performed would increase from 11 to 24. However, this would have increased the uncertainty in the results and thus a decision made to exclude these tanks from the present analysis.

6.2.5.3 Seismic Performance Data

Seismic performance data were available for all of the tanks included in the database. The performance ranged from no damage to elephant's foot buckling of the tank shell. Other forms of damage included failure of overconstrained side or bottom penetrating piping, damage to the roof or the shell top course, lateral movement of the tank or tearing of the shell (close to the bottom or near local areas of stiffness such as manholes). Only one of the tanks, Tanks N35 in Valencia, totally collapsed (Northridge Earthquake). Tables 6-1 to 6-4 also describe the types of damage for each of the tanks included in the database.

6.2.5.4 Results from Preliminary Demonstration Analysis

Eleven tanks, six located in the felt areas of the 1992 Landers Earthquake, and five in the 1994 Northridge Earthquake, were analyzed. Damage or no damage predictions (for shell buckling only) from the AWWA and Manos methodologies were compared to actual earthquake responses of the 11 tanks selected for the analysis.

All six tanks (L1, L2, L11, L12, L13 and L14, Table 6-3) selected from the Landers Earthquake were predicted to fail using the AWWA methodology, where failure is defined when the allowable stability ratio of 1.54 is exceeded. Only two of the six tanks (L1 and L2) had elephant's foot buckling and failure/tearing of the shell. Two (L12 and L13) out of the remaining four tanks had failure of overconstrained piping, which is a consequence of tank uplift or lateral movement. For these tanks, the AWWA methodology predicted uplift correctly (stability ratio greater than 0.785) but incorrectly predicted instability (stability ratio greater than 1.54), which did not occur as the tanks did not have elephant's foot buckling or tear in the shell. For Tank L11, a minor leak at the bottom flange was reported, but was unconfirmed whether it was due to a rusting plate or the earthquake. The sixth tank (L14) showed signs of lateral movement, but was not damaged.

The Manos method did not predict shell damage to five out of the six tanks (L1, L11, L12, L13, and L14). Tanks L11, L12, L13, and L14 did not have any shell damage, hence the predictions by the method were correct. However, the Manos method did not correctly predict damage to L1. This tank was located approximately 0.15 kilometers from the closest approach to the fault rupture, and there were no recorded acceleration values in the vicinity of the tank site. It is possible that the median estimates of PGA (0.56g) using the Abrahamson and Silva relationship may have under predicted the accelerations at the tank site.

The five tanks analyzed from the Northridge Earthquake were N15, N16, N17, N18, and N25 (Table 6-4) The AWWA methodology correctly predicted no damage to two tanks (N15 and N25) and also correctly predicted damage to one tank (N18). It incorrectly predicted damage to two tanks (N16 and N17) which were undamaged. The Manos method correctly predicted no damage to three tanks (N15, N16 and N25) and also correctly predicted damage to one tank (N18). Like AWWA, it incorrectly predicted damage to tank N17, which was not damaged.

In summary, the Manos method made nine correct predictions and two incorrect predictions. From the nine correct predictions, damage was correctly predicted for two tanks (L2 and N18), while no damage was correctly predicted for seven tanks (L11, L12, L13, L14, N15, N16, and N25). The AWWA method made five correct predictions (L1, L2, N15, N18 and N25) and six incorrect predictions (L11, L12, L13, L14, N16 and N17).

No final conclusions should be drawn from these demonstration analyses, because they were performed for a very small subset of the total database. However, with additional effort, some of the missing information could be readily obtained for many more tanks that would provide a better statistical sample with which to test numerical methods for assessing seismic response of tanks. The results of this additional analysis are expected to provide valuable insights in seismic performance of steel tanks that could lead to improved seismic evaluation (and possibly design) methods.

6.2.6 Mitigation of Seismic Effects

Mitigating a seismic hazard can be quite involved or relatively simple. For overconstrained piping, additional bends or a flexible section of piping may be added. Stairs and walkways can be solely supported by the tank shell. Tank wall stability can be more difficult to correct. No one method will work all the time and operating as well as construction economics should be considered.

Where the tank is found to be structurally inadequate (as determined by exceedance of the modified Manos criteria or gross exceedance of the AWWA criteria), any of the following retrofits may be implemented:

a. Reduction of the fill height; this is the simplest and most commonly recommended retrofit, and should also be considered in cases where the available freeboard is found to be inadequate. Note that reduction of fill height can have a significant effect on the economics of tank storage.

b. Increase the shell thickness and/or the bottom plate or annular ring thickness.

c. Anchor the tank in accordance with the provisions specified in AWWA D100.

d. In lieu of anchorage, prevent uplift of the tank by stiffening the tank base through the installation of a concrete slab within the tank shell, or by other methods. This method is relatively untried but may have the same effect as anchoring the tank.

6.3 TREATMENT PLANTS AND PUMP STATIONS

From an engineering standpoint, water treatment plants and pump stations present some unique challenges in addressing issues relating to seismic risk and providing mitigation alternatives (Ballantyne, 1994). They contain vastly different structures in terms of their behavior to earthquake forces. Understanding the seismic behavior of such facilities require individuals well versed in structural, geotechnical, mechanical, electrical and systems engineering.

Water treatment plants and pump stations contain structures ranging from small but highly vulnerable unreinforced masonry buildings to large complicated control buildings at the treatment plant. They contain water storage tanks with seismic behavior significantly different from that of buildings and process tanks, the latter containing delicate movable mechanical components susceptible to damage by sloshing waves. Equipment and components such as pumps, valves, electrical cabinets, control equipment and large and small diameter water and process piping require special considerations due to their inter-relationships with plant operations and their differing dynamic behavior.

Furthermore, seismic response of these facilities and components is affected by both structural and geotechnical issues. The sections below provide different types of observed seismic damage and various solutions for preventing or minimizing the damage to the various structures and components of water treatment plants and pump stations.

6.3.1 Water Treatment Plants

The primary sources of water for drinking and fire fighting purposes are either ground water or surface water. Watersheds, rivers, lakes, and impoundments are the main sources of surface water, while ground water is obtained from wells and springs. In both cases, raw water without treatment cannot be used directly for human consumption, as it contains turbidity, pathogens, organic and inorganic contaminants and dissolved solids, chemicals and minerals. In order to make water safe for drinking and remove its corrosivity, it has to be treated. Water treatment plants serve that purpose. In general water treatment plants can be divided into two main groups, ground water treatment plants and surface water treatment plants.

Ground water treatment plants are mainly used for the removal of dissolved minerals such as iron and manganese. Organic and inorganic compounds and other contaminants are also removed. Hard water is treated to make it soft in these plants.

Surface water treatment plants remove suspended solids with filtration galleries, setting basins, and clarifiers. Chemical coagulants such as alum, ferric chloride and polymers are used to facilitate removal of smaller sized suspended solids by bringing them together to join into larger heavier particles that can settle due to their own weight. Disinfection of drinking water is carried out by chlorination, sodium hypochloride, chlorine gas, ozone or potassium permangnate is typically used for this purpose. Water quality is improved by adding fluoride.

The main components of treatment plants can be divided into buildings, process tanks, equipment and piping. Each of these components and issues relating to their seismic vulnerability are discussed in the following sections.

6.3.1.1 Seismic Behavior of Buildings

Treatment plant buildings can be divided into (1) building structure and (2) non-structural elements and components. The sub sections below address these issues separately.

6.3.1.1.1 Structures

Typical buildings at a water treatment plant consist of control and operations buildings, storage buildings, maintenance buildings and chlorination and dechlorination buildings. These buildings are typically one to two story structures. Depending upon the age and location of the plant, the buildings can be of concrete masonry, steel or light metal frame construction.

Design and upgrade of these buildings should be carried out on the basis of their importance, but they should not be designed for less than the minimum code standards. Before the design or upgrade occurs, the treatment plant buildings should be prioritized in terms of their criticality. Issues such as life safety, impact on system operations, fire or chemical spill hazards and impact on adjacent facilities should be given thoughtful consideration. Total or partial collapse of operations buildings can lead to loss of life or injury to plant operators.

From a life safety standpoint, if an operations building is significantly damaged, it not only poses a threat to life safety, but it also can result in shutdown of plant and system operation. System shutdown can result in non-availability of water for fire fighting or drinking purposes. Major damage to chlorination and dechlorination buildings can result in release of chlorine or other chemicals and can be a life safety hazard. Similarly, storage sheds sometimes contain diesel tanks which, if damaged, can lead to a fire hazard.

Each facility can be rated as low, medium or high criticality. Performance criteria for each criticality rating should be established. An example of such performance criteria is as follows.

- Low Criticality – Moderate structural damage. Moderate to major non-structural damage. No partial collapse or life-threatening conditions. Shutdowns acceptable, but structural and non-structural damage repairable within months.

- Medium Criticality – Minor to moderate structural damage. Moderate non-structural damage. Limited partial shutdowns possible. Repairable within days to weeks.

- High Criticality – Minor structural damage. Minor to moderate non-structural damage. Minimal partial temporary shutdowns possible, but not probable. Structural and non-structural damage repairable within days.

Typically, current codes focus on preserving life safety during an earthquake and not continued operation. Continuity of water supply for drinking and fire fighting is critical for emergency response and recovery following an earthquake. Concepts of performance-based engineering that define levels of performance during occasional, rare and very rare seismic scenarios should be utilized (SEAOC, 1995). Generally speaking, levels of ground motion from these events have associated probabilities of exceedance of 50%, 10% and 2%, respectively, in 50 years (SEAOC, 1995). Variations from these probability levels can be justified based on a thorough evaluation of the risk levels that a community is willing to accept.

It is always prudent to avoid building types and details that have repeatedly demonstrated significant vulnerabilities in past earthquakes. One notable example is unreinforced masonry (URM) buildings. These building types do not have sufficient strength and ductility to maintain structural integrity following moderate levels of shaking. Unbraced parapets on URM buildings are particularly vulnerable and should be braced or removed. Nonductile concrete-frame buildings and tilt-up buildings have also performed poorly in historic earthquakes. Tilt-up buildings constructed prior to the mid-1970s are particularly vulnerable because of inadequate roof to wall connections which has failed during earthquakes.

Braced steel frame, Butler-type buildings are resistant to earthquakes. However, even these buildings are sometimes modified, weakening their seismic resistance. For example, bracing members have sometimes been removed or cut; such members should be repaired or replaced.

6.3.1.1.2 Non-Structural Items and Components

Improperly designed suspended ceiling panels and light fixtures are vulnerable to collapse. Light fixtures should be supported directly from fixed ceilings. Relative movement of suspended ceiling panels can sever sprinkler heads resulting in flooding. This can cause damage to computers and short circuiting of electrical controls, which can result in shutdown of operations. Ceiling panels should be connected to main structural members through diagonal wires and compression struts.

Non-structural items such as storage shelves and cabinets should be properly anchored or braced to prevent toppling or overturning.

Laboratory equipment, chemicals, and other supplies should also be secured. Office equipment and computers are vulnerable and can easily topple. They should be adequately anchored.

Raised computer floors not specifically designed for lateral seismic loading are vulnerable to collapse. One mitigation alternative is to positively anchor the raised floor to the concrete floor below and anchor computer equipment to the raised floor.

Storage shelving may topple and stored materials may fall to the floor during an earthquake. In addition to anchoring the shelving units to a load-bearing wall, lips or other types of restrainers should be provided to the front of the shelves to prevent the stored material from falling. File cabinet drawers usually roll open and fall. Latching file drawers should be used and adjacent file cabinets tied together.

6.3.1.2 Process Tanks

Process tanks at treatment plants consist of sedimentation basins whose primary purpose is to remove suspended solids through flocculation and coagulation. These tanks are rectangular, square or circular in shape and contain baffles and weirs. In most modern plants sedimentation tanks consist of clarifiers that contain moveable elements such as surface skimmers and sludge collectors.

Process tank baffles and other immersed and floating elements have been damaged from the effects of significant hydraulic loading acting on such elements. Similar damage has occurred to secondary clarifier baffles. Sloshing water has pushed rectangular access hatches out of their frames allowing them to drop to the bottom of the basin. Damaged baffles can fall to the tank bottom and must be removed before sludge collector mechanisms can be operated. Baffles and other immersed elements should be designed to either withstand the large loading or, more reasonably, to break away and be quickly replaced. Break-away elements should be secured to keep them from falling to the tank bottom and jamming sludge collector mechanisms.

Process tanks are typically constructed of reinforced concrete. They are relatively shallow in depth (approximately 10 feet). Such tanks in the United States have performed well in earthquakes. In several earthquakes outside the United States, lightly reinforced concrete baffles have failed. New concrete structures should be designed in accordance with the latest American Concrete Institute Standard *ACI 350, Environmental Engineering Concrete Structures* (ACI, 1989).

6.3.1.3 Equipment

Buildings are subjected to earthquake ground motions at their base. Due to the flexibility of the building, these motions are usually amplified over the height of the building. Largest motions are typically recorded at the roof, which can be on the order of 2 to 3 times the motions at the base. Generally, the higher in the building the equipment is located, the higher the earthquake load it experiences. Heavy equipment, such as sludge-processing equipment, should be located as low as possible in the building. Heavy equipment loads must be taken into account in the building design as it will add to the building mass resulting in larger earthquake motion forces.

Unanchored or inadequately anchored equipment can slide or topple, damaging the equipment, or causing attached piping and conduit to fail. Equipment with a low center of gravity has less of a tendency to overturn, but can still slide. In general, anchored equipment performs well, even if the anchorage is not designed for the level of anticipated seismic loading. Nonetheless, anchorage design should be performed in accordance with the latest accepted national standards. Cast-in-place anchors should be used when possible. Chemically bonded, drilled anchors are acceptable for floor installations, but can pull out in a fire. They should not be used overhead. Drilled wedge anchors are acceptable for floor and wall installations, but should not be used overhead. Drilled wedge anchors should not be used for rotating equipment, as they may work loose. Self-drilling, drilled-sleeve, or power-driven anchors should not be used.

Equipment that is supported from both sides of a building expansion joint is subject to differential support movement. Flexibility should be provided, or where possible, equipment should be supported on only one side of the expansion joint. Horizontal pumps and their motors, engines, and attached generators should be mounted on a single foundation to prevent differential movement. Vertical-turbine pumps hanging in tanks should be avoided, if possible, or designed for seismic loads.

Chlorine cylinders (150 lbs) can topple, breaking connecting piping. They should be restrained on top and bottom. Chlorine containers (1 ton) can roll or slide, breaking connecting piping and pigtails. They should be anchored with chain binders or nylon straps. Gaseous chlorine containment and stabilization measures described in the latest Uniform Fire Code (UFC) are recommended. Chlorine Institute (2001 L St. N.W., Washington, DC 20036) repair kits should be provided and stored outside the potentially hazardous area. Secondary containment should be provided for hazardous chemicals and fail-safe control devices installed on chemical feed systems. Vacuum control valves should be located as close to chlorine tanks as possible. Chlorine evaporators are particularly heavy. Often they are not installed with adequate anchorage making use of all of the bolt holes provided by the manufacturer. They should be anchored securely.

Large unanchored horizontal tanks, such as liquid natural gas, propane, diesel, or surge tanks, may slide, breaking connecting piping and draining contents. Adequate load transfer from the tank to the foundation must be provided through properly designed anchorage. Such tanks should be provided with saddles to prevent slippage and rupture of attached piping. Buried tanks and manholes may float in liquefiable material; they should be positively anchored in place.

The legs of equipment and small tanks without cross bracing may bend and collapse; bracing should be provided, as should lateral support for HVAC equipment that may otherwise fall to the floor. In some instances, failed HVAC equipment has blocked egress routes. The HVAC system may be critical for ventilation of areas with toxic fumes.

6.3.1.4 Piping

Piping and associated equipment at a water treatment plant range from large diameter influent and effluent lines to small gage chlorine piping.

Yard piping is vulnerable where it interfaces with structures, particularly when differential settlement occurs. This is of greatest concern when structures are pile supported and the piping is direct buried. Flexibility should be provided at interfaces by using double flexible couplings in series or proprietary flexible joints.

Rod-supported plant piping swings and can break at weak points, such as threaded connections and cast-iron valves. Plant piping should be supported in three orthogonal directions in accordance with the Uniform Building Code (UBC) and the Sheet Metal and Air Conditioning Contractors' National Association (SMACNA) requirements, or similar approved national standards. Equipment and pipe connections can move relative to each other and cause the system to break at the weak point. Therefore, flexible connections should be provided. Flexibility should also be provided in connections and piping where they span expansion joints or structures that can respond differentially or are on different foundations. Pipe appurtenances, such as air-release valves, respond as inverted pendulums, amplifying ground motions, and can break. They should be laterally supported.

Equipment and piping containing hazardous chemicals should be protected from falling debris. Overhead hoists should be left in a "safe" position after use so that if they start swinging, they will not break a chlorine line.

6.3.2 Pump Stations

Most pump stations are typically small single story concrete, wood or masonry structures. Due to their size and weight they are not susceptible to significant damage unless they are constructed of unreinforced masonry. Most pump stations are unmanned most of the time. Given this level of risk, a risk to life safety is, therefore, not very significant. Given this level of risk, a code based design is usually sufficient to provide adequate seismic resistance.

Below-grade pump stations are usually relatively symmetrical structures employing a heavy reinforced concrete shear wall design. They typically perform well during earthquakes (Ballantyne, 1994) provided they are not subjected to soil failures, such as liquefaction or significant differential settlement. While earthquakes may increase the lateral earth pressures on in-ground structures, the seismic inertial forces should be significantly lower in these below-grade structures than in above-grade pump-station structures.

6.3.3 Geotechnical Issues

Earthquakes will increase lateral soil pressures on in-ground structures. Methods developed by Mononobe and Okabe (Seed and Whitman, 1970) provide direction in designing for increased active soil pressures. Passive soil pressures may be activated when liquefaction-induced lateral spreading occurs.

Tank structures, channels, and large conduits in water treatment plants are vulnerable to differential settlement, increased lateral soil pressures, and flotation. Differential settlement is most likely in structures founded on soils subject to densification and/or on soils that vary significantly across the foundation. Different types of foundations across a structure also increase the risk of differential settlement. The facility should be located to avoid soils that may densify. Structures should also be located on a consistent foundation. Otherwise structures should be designed to resist expected soil failure or designed with provisions to accommodate the expected differential movement. Pounding damage or permanent movement between adjacent, but unattached or inadequately attached structures may result in the opening of expansion joints in basins. Flexibility should be provided between such structures.

Liquefaction may cause underground structures in areas of high groundwater to float or subside differentially. Keeping tanks full offers some mitigation against flotation. Landslides may also be a concern at particular sites.

6.3.4 Electrical and Communication Systems

Water treatment plants contain electric power and instrumentation systems. The plants may be served by small substations. In most all plants, local transformers are present. Other electrical and communication equipment consist of MCC and PLC cabinets, conduits and cable trays, telemetry and SCADA equipment, emergency generators and related components, and lighting.

Emergency power supplies should be provided for critical system elements because electrical power will likely be out of service following a major earthquake. To ensure they will operate, these power should be tested on a frequent basis. Priority service for the district should be arranged with the local power utility. Batteries, used for instrumentation backup and starting emergency generators, may topple if unanchored; they should be positively secured.

During an earthquake, unanchored electrical cabinets and transformers may topple or slide. These units should be anchored to the floor or attached to the wall with clips. Electric motors may be damaged from voltage-phase fluctuation. Monitoring and automatic shutdown should be provided for larger motors. Pole-mounted transformers that are unanchored may also fall, or the pole itself may topple. Emergency generators require the functioning of a series of support systems. Vibration isolators not designed for seismic loading are vulnerable. Snubbers should be used for vibration-isolated bases. The vulnerability (i.e., anchorage, support, and flexibility) of fuel, cooling, starting and exhaust systems, should be reviewed.

Telemetry systems using dedicated or hardwired systems may have had their cables broken during a major earthquake. After that event, undedicated telephone line systems will probably not have a phone line available. A radio system with adequate backup power should be provided. Electrically operated valves and equipment, or those triggered by telemetry signals, can behave unexpectedly when the power fails or the signal is lost. Plans should be developed for ways of determining the status of pumps, valves, or tank levels if the power or telemetry fails.

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TABLE 6-1: SUMMARY OF TANK PERFORMANCE DATA - LOMA PRIETA EARTHQUAKE

No.	Tank Name/ Owner	Location	Tank Physical Attributes: Capacity (MG)	Type	Diam. (ft)	Ht. (ft)	Fill Ht. During E/Q (ft)	PGA at Site (g)	Earthquake Damage: Shell Damage (Yes/No)	Description of Damage (Shell and Other)	Analysis Results: Shell Damage Predicted by AWWA (Yes/No)	Shell Damage Predicted by Modified Manos (Yes/No)	Comments
LP1	20001/Texaco	Richmond	0.840	Welded	55	48	18.5		No	None			Diesel; SBPTDM
LP2	20002/Texaco	Richmond	0.840	Welded	55	48	40.8		No	OPBP			Gasoline; SBPTDM
LP3	20003/Texaco	Richmond	0.840	Welded	55	48	26.1		No	None			Gasoline; SBPTDM
LP4	20008/Texaco	Richmond	0.840	Welded	54	49	37.6		No	Roof walkways jumped off rails			Gasoline; SBPTDM
LP5	20009/Texaco	Richmond	0.840	Welded	54	49	22.4		No	None			Gasoline; SBPTDM
LP6	20010/Texaco	Richmond	0.840	Welded	54	49	12.4		No	Roof walkways jumped off rails			Gasoline; SBPTDM
LP7	3978/Unocal	Richmond	?	Welded	42.5	?	28		Yes	EFB, SBP, fallen walkway			Lube oil; 0.3 MG storage during E/Q; SBPTDM
LP8	3957/Unocal	Richmond	?	Welded	42.5	?	28		No	OPSP			Gasoline; 0.3 MG storage during E/Q; SBPTDM
LP9	3956//Unocal	Richmond	?	Welded	42.5	?	28		Yes	SBP			Gasoline; 0.3 MG storage during E/Q; SBPTDM
LP10	3955/Unocal	Richmond	?	Welded	42.5	?	28		No	OFL			Gasoline; 0.3 MG storage during E/Q; SBPTDM
LP11	4176/Unocal	Richmond	?	Welded	50	?	33		Yes	EFB			
LP12	3949/Unocal	Richmond	?	Bolted	42.6	?	28		No	OFL			Gasoline; 0.3 MG storage during E/Q; SBPTDM

Abbreviations:
PGA - Peak ground acceleration
EFB - Elephant's foot buckling of shell
OPSP - Failure of overconstrained piping - side penetration
OPBP - Failure of overconstrained piping - bottom penetration
SBP - Failure of shell - bottom plate seam or tear in shell below manhole/side penetrating reinforced nozzle
LBF - Leakage of bottom flange (small)
OFL - Damage to shell due to overconstrained foam line
SBPTDM - Shell/bottom plate thickness data missing
RD - Roof damage
SBR - Shell buckling near roof
TLM - Tank lateral movement

Shaded areas indicate data is missing and analysis could not be performed.

TABLE 6-2: SUMMARY OF TANK PERFORMANCE DATA - COSTA RICA EARTHQUAKE

No.	Tank Name/ Owner	Location	Tank Physical Attributes					PGA at Site (g)	Earthquake Damage		Analysis Results		Comments
			Capacity (MG)	Type	Diam. (ft)	Ht. (ft)	Fill Ht. During E/Q (ft)		Shell Damage (Yes/No)	Description of Damage (Shell and Other)	Shell Damage Predicted by AWWA (Yes/No)	Shell Damage Predicted by Modified Manos (Yes/No)	
CR1	701/ RECOPE Refinery	Moin, Costa Rica	3.70	Welded	145	32	29.9(1)		No	Fire caused by Tank 792; RD			Light crude; SBPTDM
CR2	704/ RECOPE Refinery	Moin, Costa Rica	4.66	Welded	145	40	37.8(1)		No	RD			Light crude; SBPTDM
CR3	705/ RECOPE Refinery	Moin, Costa Rica	4.66	Welded	145	40	37.8(1)		No	RD			Light crude; SBPTDM
CR4	708/ RECOPE Refinery	Moin, Costa Rica	0.86	Welded	69.4	32	30.5(1)		Yes	EFB			Heavy crude; SBPTDM
CR5	709/ RECOPE Refinery	Moin, Costa Rica	0.86	Welded	69.4	32	30.5(1)		Yes	EFB			Heavy crude; SBPTDM
CR6	715/ RECOPE Refinery	Moin, Costa Rica	2.10	Welded	97.4	40	37.7(2)		No	RD			Naphtha; SBPTDM
CR7	717/ RECOPE Refinery	Moin, Costa Rica	0.76	Welded	58.6	37.5	37(2)		No	RD			Naphtha; SBPTDM
CR8	725/ RECOPE Refinery	Moin, Costa Rica	0.76	Welded	58.6	37.5	37(2)		No	RD			Heavy naphtha; SBPTDM
CR9	726/ RECOPE Refinery	Moin, Costa Rica	0.76	Welded	58.6	37.5	37(2)		No	RD;TLM			Heavy naphtha; SBPTDM
CR10	728/ RECOPE Refinery	Moin, Costa Rica	4.07	Welded	134	40	38.6(1)		No	SBR; TLM			Bunker oil; SBPTDM
CR11	Unknown/ RECOPE Refinery	Moin, Costa Rica	3.95	Welded	134	40	37.5(1)		No	TLM			Diesel; SBPTDM
CR12	738/ RECOPE Refinery	Moin, Costa Rica	0.42	Welded	48	32	31.1(3)		Yes	EFB			Gasoline; SBPTDM
CR13	745/ RECOPE Refinery	Moin, Costa Rica	0.21	Welded	34	32	31.0(3)		Yes	EFB			Diesel; SBPTDM
CR14	792/ RECOPE Refinery	Moin, Costa Rica	0.023	Welded	15.7	15.9	15.9(1)		Yes	Overturned tank; explosion			Slop; SBPTDM

TABLE CONTINUED ON NEXT PAGE

TABLE 6-2: Continued

Abbreviations:	EFB	-	Elephant's foot buckling of shell
	OPSP	-	Failure of overconstrained piping - side penetration
	OPBP	-	Failure of overconstrained piping - bottom penetration
	SBP	-	Failure of shell - bottom plate seam or tear in shell below manhole/side penetrating reinforced nozzle
	LBF	-	Leakage of bottom flange (small)
	OFL	-	Damage to shell due to overconstrained foam line
	SBPTDM	-	Shell/bottom plate thickness data missing
	RD	-	Roof damage
	SBR	-	Shell buckling near roof
	TLM	-	Tank lateral movement

Notes:	(1)	Estimated fill height based on tank being reportedly full at time of earthquake
	(2)	Tank assumed to be full based on fluid sloshing damage
	(3)	Tank assumed to be full based on observed damage (note that this may or may not be the case)

Agencies: RECOPE - Recope Oil Refinery (Costa Rican oil refinery, Refinadora Costarricense de Petroleo)

Shaded areas indicate data are missing and analysis could not be performed.

TABLE 6-3: SUMMARY OF TANK PERFORMANCE DATA – LANDERS EARTHQUAKE

No.	Tank Name/ Owner	Location	Tank Physical Attributes					PGA at Site (g)	Earthquake Damage		Analysis Results		Comments
			Capacity (MG)	Type	Diam. (ft)	Ht. (ft)	Fill Ht. During E/Q (ft)		Shell Damage (Yes/No)	Description of Damage (Shell and Other)	Shell Damage Predicted by AWWA (Yes/No)	Shell Damage Predicted by Modified Manos (Yes/No)	
L1	8/BDVWA	W. of Landers	0.42	Welded	55	24	21.9	0.56	Yes	EFB, OPSP, SBP	Yes	No	
L2	CSA-70	Landers	0.21	Bolted	38.6	24.1	22.0	0.47	Yes	EFB, OPSP, SBP	Yes	Yes	Assumed fill ht. = 22'; Assumed Subsurface is soil
L3	"B"/BDVWA(1)	W. of Landers	0.10	Welded	26.5	24	22.8	0.55	No	None			SBPTDM
L4	"C"/BDVWA(1)	W. of Landers	0.50	Welded	59.5	24	22.6	0.55	No	None			SBPTDM
L5	10/BDVWA	W. of Landers	0.10	?	32.6	16	14.6	0.55	No	None			SBPTDM
L6	22A/BDVWA	Flamingo Hts.	0.10	?	32.6	16	14.6	0.54	No	None			SBPTDM
L7	22B/BDVWA	Flamingo Hts.	0.10	?	32.6	16	14.6	0.54	No	None			SBPTDM
L8	22C/BDVWA	Flamingo Hts.	0.20	?	46	16	14.6	0.54	No	None			SBPTDM
L9	22D/BDVWA	Flamingo Hts.	0.50	?	73	16	14.5	0.54	No	None			SBPTDM
L10	3H/BDVWA	S. of Flamingo Hts.	0.042	?	21	16	14.7	0.55	No	None			SBPTDM
L11	Beryl/SCWC	Barstow	0.13	Bolted	30	24	21	0.14	No	LBF	Yes	No	
L12	Basalt/SCWC	Barstow	0.13	Bolted	30	24	21	0.14	No	OPBP	Yes	No	
L13	Arville-N/SCWC	Barstow	0.20	Welded (fillet)	29.3	41.5	37	0.14	No	OPBP	Yes	No	
L14	Arville-S/SCWC	Barstow	0.22	Welded	29.3	44.5	40	0.14	No	TLM	Yes	No	

TABLE CONTINUED ON NEXT PAGE

TABLE 6-3: Continued

Abbreviations:	EFB	-	Elephant's foot buckling of shell
	OPSP	-	Failure of overconstrained piping - side penetration
	OPBP	-	Failure of overconstrained piping - bottom penetration
	SBP	-	Failure of shell - bottom plate seam or tear in shell below manhole/side penetrating reinforced nozzle
	LBF	-	Leakage of bottom flange (small)
	OFL	-	Damage to shell due to overconstrained foam line
	SBPTDM	-	Shell/bottom plate thickness data missing
	BPTDM	-	Bottom plate thickness data missing
	RD	-	Roof damage
	SBR	-	Shell buckling near roof
	TLM	-	Tank lateral movement
Notes:	(1)		Precise name not known; two tanks at this location (currently designated Tanks "B" and "C")
Agencies:	BDVWA	-	Bighorn Desert View Water Agency
	CSA	-	County Service Agency
	SCWC	-	Southern California Water Company

Shaded areas indicate data are missing and analysis could not be performed.

TABLE 6-4: SUMMARY OF TANK PERFORMANCE DATA – NORTHRIDGE EARTHQUAKE (Page 1 of 3)

No.	Tank Name/ Owner	Location	Tank Physical Attributes					PGA at Site (g)	Earthquake Damage		Analysis Results		Comments
			Capacity (MG)	Type	Diam. (ft)	Ht. (ft)	Fill Ht. During E/Q (ft)		Shell Damage (Yes/No)	Description of Damage (Shell and Other)	Shell Damage Predicted by AWWA (Yes/No)	Shell Damage Predicted by Modified Manos (Yes/No)	
N1	Alta Vista/LADWP	San Fernando Valley	0.5	Riveted	54	30	29	0.90	No	None			BPTDM
N2	Alta Vista/LADWP	San Fernando Valley	1.8	Welded	95	36.5	30.5	0.90	No	None			BPTDM
N3	Alta View/LADWP	San Fernando Valley	1.0	Riveted	65	42.5	41	0.25	No	STL			SBPTDM
N4	Beverly Glen/LADWP	San Fernando Valley	2.2	Riveted	100	40.604	Unknown	0.28	No	SBR, RD, OPSP			BPTDM
N5	Clearwell/LADWP	San Fernando Valley	4.6	Welded	140	40	38.3	0.96	No	None			SBPTDM
N6	Coldwater/LADWP	San Fernando Valley	2.2	Riveted	100	40.604	Unknown	0.16	No	RD, TLM, OPSP			BPTDM, foundation settlement
N7	Corbin/LADWP	San Fernando Valley	4.0	Welded	156	30	25	0.38	No	OPBP			Partially buried, BPTDM
N8	Donick/LADWP	San Fernando Valley	1.1	Welded	122.8	24	22.5	0.38	No				SBPTDM
N9	Granada High/ LADWP	San Fernando Valley	0.58	Riveted	55	35	31.7	0.46	Yes	C, OPSP			Tank Removed, SBPTDM, drwgs. Available
N10	Kittridge 3/LADWP	San Fernando Valley	10.0	Welded	190	51	Unknown	0.40	No				1" annular ring, BPTDM
N11	Kittridge 4/LADWP	San Fernando Valley	10.0	Welded	190	51	Unknown	0.40	No				1" annular ring, BPTDM
N12	Mulholland/LADWP	San Fernando Valley	0.52	Riveted	52.5	35	Zero	0.23	No	Overflow pipe damage			Tank out of service during EQ
N13	Topanga/LADWP	San Fernando Valley	0.215	Welded	18	29.7	26.5	0.38	No	OPSP			BPTDM
N14	Zelzah/LADWP	San Fernando Valley	1.0	Welded	70	40	32.3	0.22	Yes	OPSP, RD, EFP (minor)			BPTDM
N15	Alamo/SCWC	Simi Valley	1.5	Welded	100	26	20.5	0.30	No	None	No	No	
N16	Crater East/SCWC	Simi Valley	0.5	Bolted	60	24.1	Unknown	0.30	No	None	Yes	No	Fill ht. at time of earthquake assumed to be 22'
N17	Crater West/SCWC	Simi Valley	0.21	Bolted	40	24.1	23	0.30	No	None	Yes	Yes	Fill ht. At time of earthquake assumed to be 23'
N18	Katherine/SCWC	Simi Valley	0.21	Bolted	38.67	24.1	20-21	0.75	Yes	EFB, OPSP, SBR, RD, LBF	Yes	Yes	BPTDM

TABLE 6-4 CONTINUED (Page 2 of 3)

N19	Lautenschlager #1/SCWC	Simi Valley	0.5	Welded	64	22	19.5		No	None			BPTDM
N20	Lautenschlager #2/SCWC	Simi Valley	0.5	Welded	64	24	19.5		No	None			BPTDM
N21	Rebecca North/SCWC	Simi Valley	0.21	Bolted	38.67	24.1	22-23	0.32	Yes	EFB, OPSP, SBR			BPTDM
N22	Rebecca South/SCWC	Simi Valley	0.126	Bolted	29.75	24.1	22-23	0.32	Yes	OPSP, EFB, SBR			BPTDM, onset of EFB
N23	Sycamore North/SCWC	Simi Valley	0.126	Bolted	29.75	24.1	16.5	0.30	Yes	OPSP, EFB, LBF			BPTDM, onset of EFB
N24	Sycamore South/SCWC	Simi Valley	0.21	Bolted	38.67	24.1	16.5	0.30	Yes	EFB			BPTDM, onset of EFB
N25	Tapo/SCWC	Simi Valley	3.0	Welded	130	32.1	28.5	0.32	No	None	No	No	
N26	Newhall No. 1/NCWD	Newhall	0.63	Welded	60	30	Unknown		Yes	EFB, C, OPSP			SBPTDM
N27	Newhall No. 2/NCWD	Newhall	0.3	Welded	40	32	Unknown		Yes	EFB, OPSP, TLM			SBPTDM, damage to tub ring
N28	Newhall No. 3/NCWD	Newhall	0.3	Welded	40	32	Unknown		Yes	OPSP, TLM, SBP			SBPTDM, damage to tub ring
N29	Newhall No. 4/NCWD	Newhall	0.3	Welded	40	32	Unknown		Yes	OPSP, OPBB, TLM, SBP			SBPTDM, damage to tub ring
N30	Newhall No. 5/NCWD	Newhall	0.75	Welded	64	32	Unknown		Yes	OPSP, OPBB, TLM, SBP			SBPTDM, damage to tub ring
N31	Newhall No. 6/NCWD	Newhall	0.06	Welded	20	20	Unknown		Yes	OPSP, OPBB, EFB, SBP			SBPTDM, damage to tub ring
N32	Newhall No. 7/NCWD	Newhall	1.5	Welded	90	32	Unknown		No	RD, SBR			SBPTDM, evidence of rocking
N33	Amir/unknown	Simi Valley	0.3	Welded	42	29.83	Unknown		Yes	EFB			Fire protection tank, BPTDM
N34	Larwin	Santa Clarita	0.82	Welded	60	40	32		Yes	EFB, SBP, RD			SBPTDM, anchored tank
N35	MM1	Valencia	0.5	Bolted	60	24	20		Yes	C			SBPTDM
N36	MM2	Valencia	0.75	Bolted	73	24	20		No	Damage			Damage due to adjacent MM1 collapse, SBPTDM
N37	Santa Clarita	Valencia	1.5	Welded	80	40	39		Yes	EFB, RD			SBPTDM, location & owner unknown
N38	A/Sepulveda Terminal	Sepulveda	0.84	Welded	65	36	24		No				SBPTDM, Fuel storage tank
N39	B/Sepulveda Terminal	Sepulveda	1.05	Welded	72	36	12		No				SBPTDM, Fuel storage tank
N40	C/Sepulveda Terminal	Sepulveda	0.76	Welded	60	36	12		No				SBPTDM, Fuel storage tank
N41	1/Van Nuys Terminal	Van Nuys	0.23	Welded	29	48	Unknown		No				SBPTDM, Fuel storage tank

TABLE 6-4 CONTINUED (Page 3 of 3)

N42	2/Van Nuys Terminal	Van Nuys	0.36	Welded	36	48	Unknown		No				SBPTDM, Fuel storage tank
N43	3/Van Nuys Terminal	Van Nuys	1.26	Welded	67	48	Unknown		No				SBPTDM, Fuel storage tank
N44	4/Van Nuys Terminal	Van Nuys	1.43	Welded	72	48	Unknown		No				SBPTDM, Fuel storage tank
N45	5/Van Nuys Terminal	Van Nuys	0.038	Welded	15	30	Unknown		No				SBPTDM, Wastewater

Abbreviations:
- C - Collapse
- EFB - Elephant's foot buckling of shell
- OPSP - Failure of overconstrained piping - side penetration
- OPBP - Failure of overconstrained piping - bottom penetration
- SBP - Failure of shell - bottom plate seam or tear in shell below manhole/side penetrating reinforced nozzle
- LBF - Leakage of bottom flange (small)
- \- Roof damage
- SBR - Shell bucking near roof
- STL - Settlement
- TLM - Tank lateral movement
- BPTDM - Bottom plate thickness missing
- SBPTDM - Shell and bottom plate thickness data missing Agencies:
- LADWP: - Los Angeles Department of Water and Power

Shaded areas indicate data are missing and analysis could not be performed.

NCWD - Newhall County Water District

SCWC - Southern California Water Company

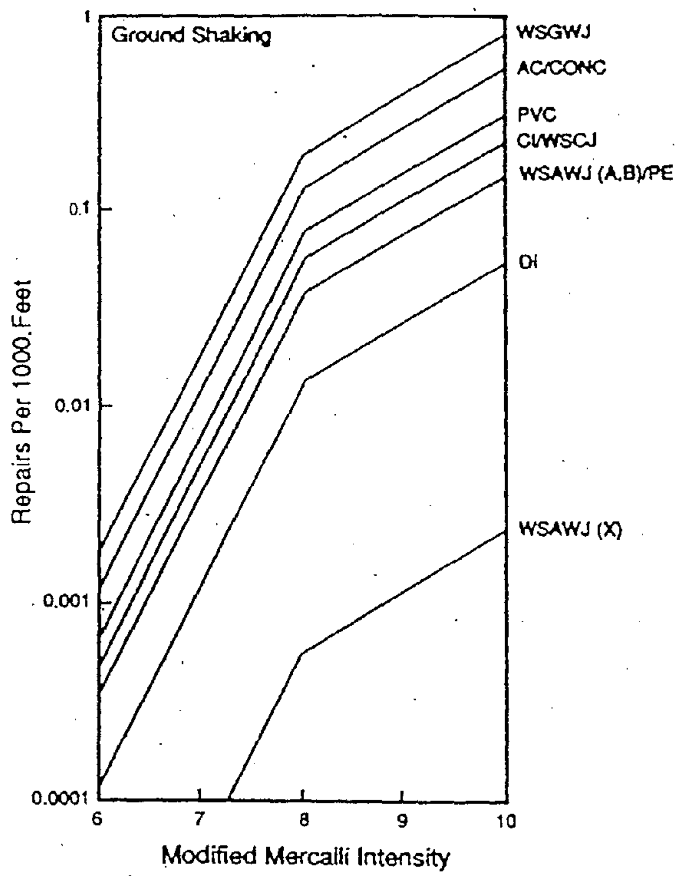


Figure 6-1 Pipeline Repair Rates for Different Pipes Versus Modified Mecalli Intensity (Eguchi, 1983)

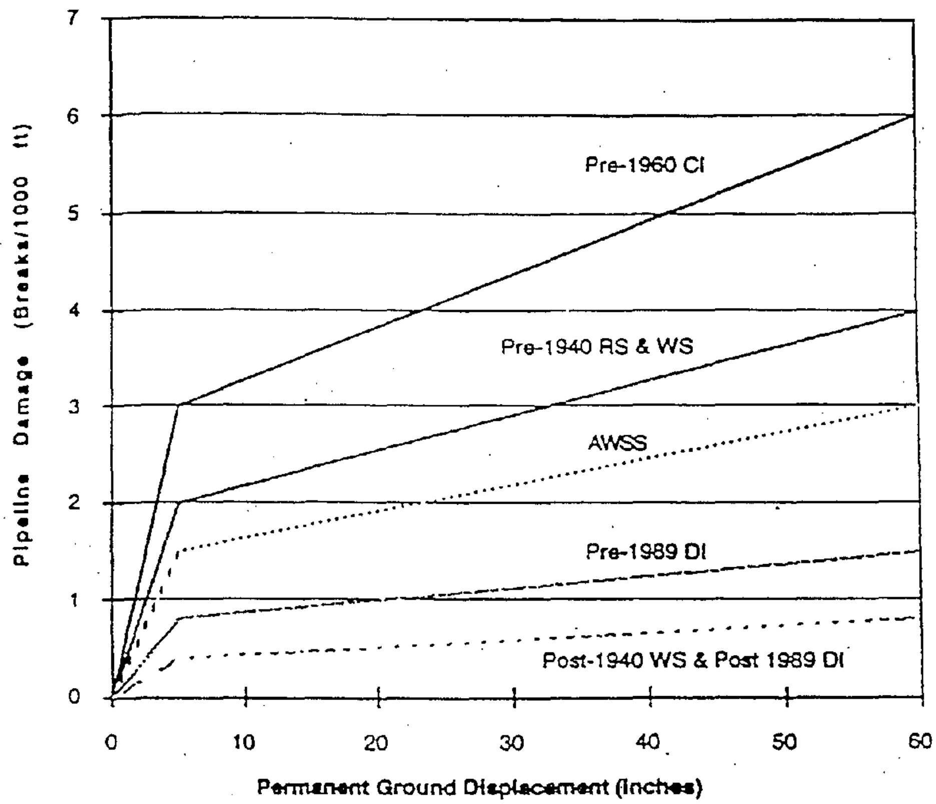


Figure 6-2 Pipeline Damage Rates for Different Pipe Versus Permanent Ground Displacement (Harding-Lawson et al, 1991)

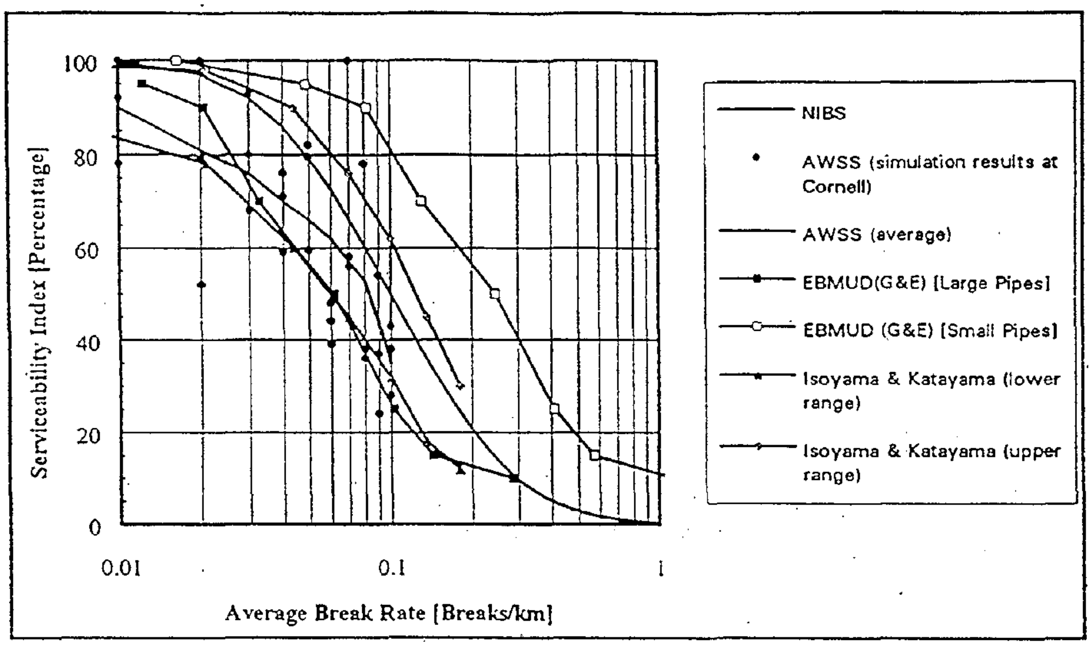


Figure 6-3 Damage Index Versus Average Break Rate for Post-Earthquake System Performance Evaluation (RMS, 1996)

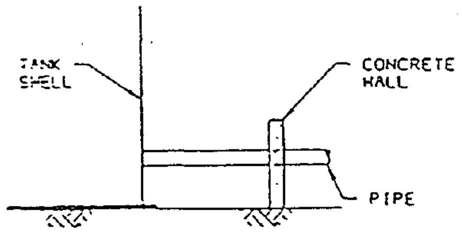
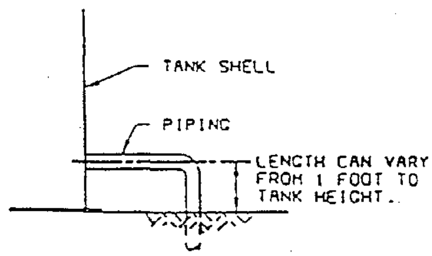
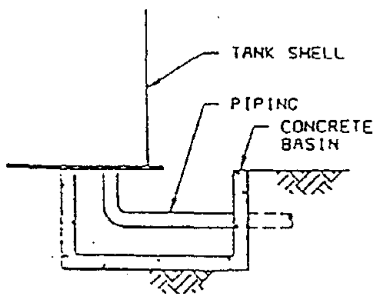
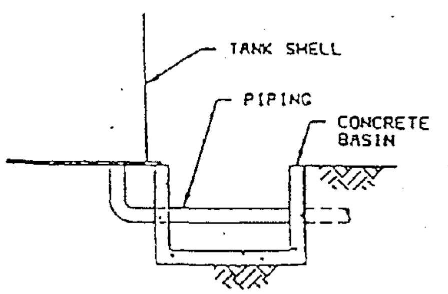
Poor Details	Retrofit Recommendations
(a) 	• Add flexibility to pipe
(b) 	• Add flexibility to pipe
(c) 	• Add flexibility to pipe
(d) 	• Reroute piping to center of tank or extend concrete basin beyond pipe/tank connection

Figure 6-4 Typical Poor Details at Unanchored Tanks and Retrofit Recommendations (Adapted from Dowling and Summers, 1993)

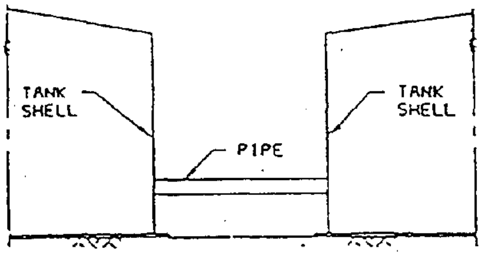
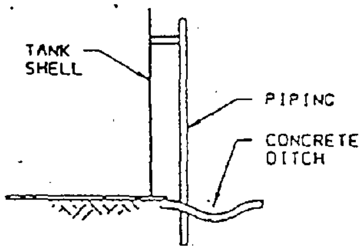
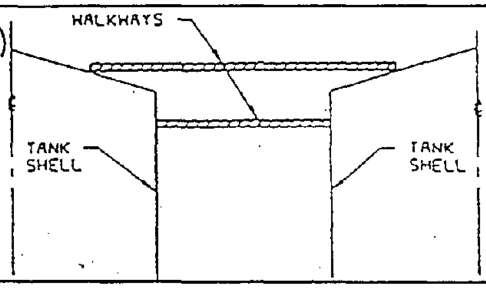
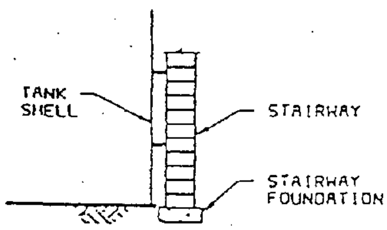
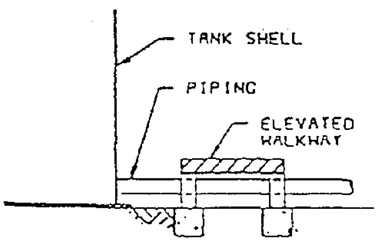
Poor Details	Retrofit Recommendations
(e) 	• Increase flexibility by providing horizontal or vertical bends
(f) 	• Anchor pipe at shell roof connection
(g) 	• Increase walkway flexibility to accommodate relative displacements
(h) 	• Support stairway exclusively on tank shell
(i) 	• Increase piping flexibility, attach walkway exclusively to tank shell, or provide more piping clearance

Figure 6-4(Cont.) Typical Poor Details at Unanchored Tanks and Retrofit Recommendations (Adapted from Dowling and Summers, 1993)

7.0 MITIGATION ALTERNATIVES

7.1 WORKING RELATIONSHIPS BETWEEN FIRE DEPARTMENTS AND WATER AGENCIES

7.1.1 Recommendations to open communications between fire and water departments

Fire and water departments bear joint responsibility in disasters involving fires and the delivery of adequate water supply to control the fires that occur. If mains are inadequate, or are gated to set up different zones, these gates must be opened at such major fire events to insure adequate supply to fire department pumpers.

Communications between the two agencies is critical to allow for coherent operations of the two agencies at emergencies. Fire Departments and Water Agencies should meet and agree upon protocols and operating procedures for operations at emergencies.

Communications by a water department representative to the Incident Commander at a fire incident should be defined and structured so it fits into the overall emergency operations procedures. Fire Department command and dispatch centers should be tasked with contacting the Water Department Operations Center to respond to a fire incident. Major incidents such as earthquakes and wild land fires will demand the utmost of water departments to maintain adequate fire flows. Satisfactory communications between the two agencies will be a critical factor in satisfactory operations.

A standard operating procedure could be developed where the fire command center automatically sends a request for a water agency representative to respond to the incident. A recommended response would be for water department response to all second or greater alarms that are struck for a fire. A second alarm or greater is defined as the call for assistance beyond the initial fire response to a fire call.

With such an automatic response procedure for large fires, fire dispatch personnel would get used to the routine of calling for the water department response and the water department personnel will be used to the calls and reporting into the command post at actual incidents.

It is important that this response procedure of water department personnel to large fires be structured as automatic response, it should not be seen as an extraordinary duty for water department personnel, nor should it be left to Incident Commanders to request water department representatives to respond. The command post at a major fire can become very hectic and a request for a water supply representative could fall through the cracks in requesting response. If water supply problems develop, there can be considerable delay in the response of the water supply representative if they are requested after the event has occurred.

7.1.2 Post earthquake issues facing fire and water departments

Earthquakes can cause major disruption in water supply systems. This can have catastrophic results in fire fighting following an earthquake where the water system suffers substantial damage. Fires following earthquake occurs with regularity in built up areas due to a variety of ignition sources. The first priority of fire departments following earthquakes is to quickly determine where fires have occurred and to devise an action plan to control them.

- **Water supply will be a primary concern for the fire department in this situation.** The first priority of water departments following an earthquake is to determine where breaks have occurred in mains or storage facilities and what the situation is regarding electric power to operate pumps and other facilities.

- **Major breaks must be quickly isolated so that reservoirs are not drained.** Coordination with fire departments must take place so that water supply can be provided or maintained where fires have occurred.

- **Communications between fire and water departments will be critical in the initial phase of the post earthquake event.**

Post earthquake scenarios where damage to the water system and fire breakouts have occurred can have catastrophic consequences for a city or urban area. The primary mission of the fire department will be to try to prevent conflagration conditions developing with the fire situation. Water supply to allow the fire department to function will be the critical issue in success or failure of fire fighting efforts.

Pre-event planning will be of utmost importance for successful emergency operations of fire and water departments. Actions by employees of both department will have to be taken that will be in effect, "damage control", to enable systems to function initially. The importance of an operations plan for actions following earthquake by both departments is critical.

Training exercises, both table top and actual field exercises in closing down valves, communications and coordination must be done from time to time to insure that plans and procedures that have been developed actually work in a practical manner. Both fire and water agencies should participate in such exercises, with critiques to be held following such joint training exercises.

After the fires are brought under control, the fire and water departments will then focus their efforts in getting the damaged water system back into service as soon as possible. This may entail fire departments assisting water departments in using their equipment to help in bypassing breaks and getting temporary systems in place to provide fire flows until permanent repairs are made to underground mains.

As an example, following the Northridge earthquake in southern California, fire departments worked with the water districts for several weeks in providing assistance in pumping from one water zone to another, bypassing breaks and restoring service to undamaged areas that were gated off.

Pre-planning by fire and water departments for earthquake events can usually establish the most likely areas of the water system that will sustain damage. Study of soil conditions and engineering studies of the water system can usually develop an accurate profiles of where damage is most likely to occur to the water system. Pre-planning to mitigate this damage can be developed and emergency operations plans can be developed to deal with this projected scenario. This type of planning and coordination between fire and water departments is very important for them to have some idea of the difficulties they will be facing in such events. Contingency planning for both departments in these situations will have great benefit for successful operations.

This type of planning will be critical for the fire department in understanding the potential of large fires that may develop into conflagrations. The fire service needs to know the weak points of the water system and to be able to anticipate where breaks in mains are likely to occur. With this knowledge they can develop fire fighting operational plans that will be realistic under the conditions that they will be facing.

Restoration of the water lifeline systems following an earthquake is a top priority for fire and water departments. Planning, communications, coordination between both agencies is vital and necessary for successful emergency operations. Each agency must be aware of the needs and problems facing the other, as without water supply the dimension of the disaster can reach catastrophic proportions.

7.2 PRIORITIZATION OF SYSTEM HARDWARE REPLACEMENT

The prioritization of system hardware replacement consists of a three step procedure, as described below:

(1) In the first step, each component of the system is rated in terms of its criticality with respect to system operation, with the main objectives consisting of life safety, fire fighting, providing emergency water, and being able to complete system restoration following an earthquake. Component criticality refers to the significance of a given component with respect to these issues. By way of example, an operations building is more critical than an unmanned tool storage shed. The criticality of each component is established in terms of an overall criticality rating (OCR).

(2) The second step consists of an engineering evaluation of each component of the system to establish its seismic vulnerability by estimating expected damage that the component may experience during the postulated earthquake. The expected seismic performance of a structure or component against an established performance criterion is known as the seismic vulnerability. An attempt should be made to assign seismic vulnerability ratings (SVRs) to all facilities in the system.

(3) The final step in the prioritization process consists of assigning a retrofit priority to each of the components of the system, with consideration given to both the criticality and vulnerability.

These concepts are described in more detail below.

7.2.1 Development of Overall Criticality Rating (OCR)

Overall criticality ratings (OCRs) are assigned to each component and pipeline segment in the system in numerical order from 5 (highly critical) to 1 (not critical). The OCR is generally chosen to be a weighted average of the following six individual criticality ratings (or any other criticality ratings, as appropriate), in which each individual criticality rating also ranges from, say, 5 (highly critical) to 1 (not critical):

SOR - System Operation Rating
CSR - Capacity/Size Rating
LSR - Life Safety Rating
FFR - Fire Flow Rating
DWR - Drinking Water Rating
DPR - Damage Potential Rating

The first two individual criticality ratings above (SOR and CSR) can be combined to form a Facility Criticality Rating (FCR) using Equation (7-1) below. The FCR remains the same for each component at any given facility. For example, a clarifier and a motor control center (MCC) at a given treatment plant should both have the same FCR. The FCR is intended to establish the importance of each facility in the system. As another example, two large tanks at different locations in the system that may have the same CSR, may very likely have different SORs (since they may be of different criticality with respect to system operations), and hence, have different FCRs.

$$FCR = (X_1 \times SOR) + (X_2 \times CSR) \tag{7-1}$$

The latter four individual criticality ratings described above (LSR, FFR, DWR and DPR) can be combined using Equation (7-2) to form a Component Criticality Rating (CCR) for each of the components at any given facility. In the above example, the clarifier and the MCC at the treatment plant need not have the same CCR.

$$CCR = (Y_1 \times LSR) + (Y_2 \times FFR) + (Y_3 \times DWR) + (Y_4 \times DPR) \tag{7-2}$$

Finally, the FCR and CCR can be combined using Equation (7-3) below to give the Overall Criticality Rating (OCR).

$$OCR = (Z_1 \times FCR) + (Z_2 \times CCR) \tag{7-3}$$

The relative weights assigned to each of the individual criticality ratings described above need to be based on individual system performance requirements. The weighting factors for the FCR (X_1, X_2), the CCR (Y_1, Y_2, Y_3, Y_4) and the OCR (Z_1, Z_2), should add respectively to 1.0.

An example definition of the individual component criticality ratings is given below.

<u>System Operation Rating</u> (SOR) ranges from 5 (highly significant with respect to system operation) to 1 (not highly significant with respect to system operation).

<u>Capacity/Size Rating</u> (CSR) is a function of component size. As an example, for reservoirs, the CSR could be defined as follows:

5 - 5,000,000 gallons or greater
4 - 3,000,000 - 5,000,000 gallons
3 - 1,500,000 - 3,000,000 gallons
2 - 750,000 - 1,500,000 gallons
1 - Less than 750,000 gallons

For other components, the CSR can be based on relative water throughput.

<u>Life Safety Rating</u> (LSR) can be assigned as follows:

5 - Continuously occupied
4 - Hazardous materials release
3 - Occupied 50% of time
2 - Occupied 25% of time
1 - Occupied 10% of time or less

<u>Fire Flow Rating</u> (FFR) ranges from 5 (highly critical with respect to providing water for fire suppression) to 1 (not critical with respect to providing water for fire suppression).

Drinking Water Rating (DWR) ranges from 5 (highly critical with respect to providing drinking water following an earthquake) to 1 (not critical with respect to providing drinking water following an earthquake).

Damage Potential Rating (DPR) ranges from 5 (significant issues with respect to damage potential to adjacent facilities, e.g., numerous residences in the direct flood path of a potential catastrophic reservoir failure) to 1 (minimal damage potential to adjacent facilities). The DPR can also reflect the damage potential to the component itself, adjacent facilities or the entire system, e.g., an unrestrained bookshelf that could fall and adversely impact critical computer/control equipment causing significant damage should have a high DPR.

Pipeline segments should also be assigned an OCR that ranges from 5 (highly critical) to 1 (not critical). Pipeline OCRs are usually established in a discussion with the utility personnel. The OCRs selected for pipeline segments can be the same as the SORs for the various facilities along the pipeline system. OCR 5 pipelines can be those required to move water to and from major portions of the system. There is usually no redundancy for OCR 5 pipelines. OCR 4 pipelines, on the other hand, often have some limited redundancy, but are still critical "backbone" transmission pipelines. OCR 3 pipelines are those required to serve the various cities along the extremities of the system. Each OCR 3 pipeline serves only a relatively small area. OCR 1 and 2 pipelines are generally only for distribution systems, rather than the backbone transmission system.

7.2.2 Determination of Seismic Vulnerability Rating (SVR)

7.2.2.1 Facilities

Seismic vulnerability ratings (SVRs) should be assigned to each component based on the seismic performance estimates during the postulated (design or evaluation level) earthquake. SVRs range from 5 to 1 in accordance with the ratings given in Table 7-1. SVRs are assigned if one or more of the criteria in Table 7-1 is met.

7.2.2.2 Pipelines

SVRs can be assigned to each pipeline segment based on the seismic performance estimates in terms of damage rate (damages per km) for the postulated earthquake, where damage is defined as a pipeline leak or break that would be identified within 7 days following the earthquake. SVRs can range from 5 to 1 as shown in Table 7-2.

7.2.3 Retrofit Prioritization

Retrofit prioritization is accomplished through use of a risk analysis matrix relating the seismic vulnerability rating, SVR, and the overall criticality rating, OCR. An example is provided in Table 7-3.

The significance of the recommended action items in each of the retrofit categories, A, B, C, and D, should be evaluated, including estimated cost and consequences to the system if the component or pipeline segment was to fail. All priority A and B recommendations should generally be implemented. Priority C and D recommendations should be evaluated on their own merit.

7.3 MONITORING AND CONTROL OF LIFELINE SYSTEMS AS A MITIGATION ALTERNATIVE

7.3.1 Introduction

This section summarizes the problem of earthquake damage to lifeline systems, particularly buried pipe, and the high cost of mitigation by replacement. System monitoring and control is presented as an alternative. Earthquake hazard, structural, soils, and system operation parameters are identified as useful for system control; examples are presented. Monitoring and control system implementation issues are discussed including system configuration, local/centralized control, hardware, and appropriate types of systems for earthquake mitigation implementation.

7.3.2 Problem Statement

Earthquakes cause damage to lifeline systems, particularly buried pipelines and transportation facilities. Upgrade of these lifeline system elements to remain functional following design level earthquakes can be prohibitively expensive, and may not be technically feasible.

Examples of lifeline system failures include water and gas systems in both the Northridge, California, and Kobe, Japan earthquakes. The Northridge Earthquake initiated 58 building structure fires, 51 of which involved natural gas; 172 mobile homes were destroyed by fire, most fueled by natural gas. An estimated 100,000 services were without water as a result of failure of 24 of LADWP's major trunk lines and 700 distribution lines. Fire crews were using tankers to transport water to fires. A ruptured liquid fuel line discharged 4,120 barrels of product into the Santa Clara River requiring a major cleanup effort.

In Kobe, Japan, approximately one million houses were without water following the January 17, 1995 event as a result of leakage from 4,200 pipeline failures. Two-thirds of the reservoir sites serving urban Kobe, providing water for drinking and fire protection, drained within six hours. There were over 148 fire ignitions; approximately one-third from natural gas. It took six hours to shut down the gas distribution system.

Wholesale replacement of water system pipeline materials vulnerable to earthquakes is prohibitively expensive as an earthquake mitigation measure. The Seattle water system has approximately 2,400 km of pipelines less than 31 cm in diameter and approximately 550 km of pipelines larger than 31 cm in diameter (Ballantyne, 1988). Approximately 6 percent is in areas highly vulnerable to liquefaction. Most of the system is cast iron pipe. The estimated cost to replace the entire system with ductile iron or welded steel pipe would be over $1.5 billion. Replacing just the pipelines in areas susceptible to liquefaction would cost an estimated $100 million. These costs exceed the Seattle Water Department's financial capability to replace vulnerable pipelines. A monitoring and control system alternative will mitigate the effects of some pipeline damage at a cost less than replacing vulnerable pipelines.

There is some question whether even pipelines constructed with the most seismic resistant materials and designs will resist large permanent ground deformation.

7.3.3 Post-earthquake System Monitoring and Control Strategy

The mitigation strategy described in this paper is to monitor 1) seismic hazard parameters, 2) structural component and soil parameters, and 3) post-earthquake system operation. With that information, control secondary damage that might occur and/or keep systems functional to provide continued service. Secondary damage that might occur includes trains running off the tracks where embankments or bridges have failed, or natural gas from broken pipelines igniting. Continued service could be maintained in water systems by isolating damaged parts of the system that would then allow provide water for fire suppression in undamaged areas.

Evaluation of seismic hazard parameters such as peak ground acceleration would enable identification of areas of high peak ground accelerations where structural damage and soil failures are more likely to occur. Table 7-4 shows examples of monitoring and control approaches.

One or more of these parameters can be combined to provide a more reliable assessment of the situation. In water systems, peak ground acceleration, PGA, flow, rate of change of flow rate, and/or pressure can be combined for various levels of controlled response. For gas service meters, flow, flow duration, seismic intensity, pressure, and gas concentration exceeding predetermined levels have are all monitored to make a shutdown decision.

PGA threshold levels for earthquake valves on reservoirs in the Kobe, Japan water system are set at the following levels with the corresponding action:

40 gal: manual alarm (where 980 gal = 1 x gravity)
80 gal combined with rate of change of flow: shut down
250 gal: shut down

Standard Japanese trains are directed to slow to 15 kph with PGAs exceeding 40 gal, until the first train passes and does not identify damage. At 80 gal, train service is suspended, and trains running on tracks are directed to proceed to the nearest station at 15 kph. The Shinkansen (Bullet Train) power is shut down at a 40 gal threshold, with resumption of service at reduced speeds and/or rerouting depending on the earthquake magnitude and intensity, as measured by seismographs at each transformer station. All bullet trains are rerouted when the PGA exceeds 120 gal. There is increasing concern with quick shut down as train speeds increase to 270 kmph, requiring 3 to 4 km to stop.

Measuring offsets of structural component differential movement is being used both for building of active control systems and tunnels. The technology is available for monitoring pipe joint displacement, but is probably too expensive for any but the most crucial pipelines and those serving as surrogates of long pipeline segments. In-situ liquefaction monitoring technology has been developed and implemented by Tokyo Gas; implementation is unknown.

System operation parameters, particularly flow, is being used both independently as well as in combination with shaking intensity as a control parameter.

Ground motion instrument distribution can have a significant effect on the level of accuracy of ground motions at any given site. Kobe Water has a single ground motion instrument centrally located within there system that is used for post earthquake control of the system. Osaka Gas has approximately 30 ground motion instruments that are integrated to calculate ground motions throughout their system. Standard Japanese railways locate ground motion instruments approximately every 40 km along their alignment, with the Shinkansen locating them every 25 km. The USGS/California Institute of Technology CUBE system makes use of the USGS network of ground motion instruments.

7.3.4 Applications in Different Water System Configurations

Further discussion in this section focuses on application of monitoring and control of water systems. There are three basic applications for monitoring and control (isolation valves) in water systems, 1) reservoir isolation, 2) pipelines vulnerable at rivers and/or fault crossings, and 3) areas susceptible to liquefaction. In all three cases, it is the intent to isolate the stored water from the damaged pipeline system. Another application is the use of an isolation valves on one of two reservoirs at a single site, maximizing system reliability, but still isolating some water from damaged pipelines. This application was in place in Kobe, Japan, and worked effectively following the earthquake.

7.3.5 System Control and Keeping the System Operating

Water systems can be monitored and controlled either locally, or from a central location. In a pure, local system, the parameters selected for control would be monitored, when the set point was reached, the isolation valve would be actuated. Alternatively, parameters such as ground motion could be monitored locally and telemetered back to a central location; parameters such as ground motion could be monitored at the central location. Control decisions could then be made in the central location where trained staff could evaluate other system conditions and make a more informed decision. The advantage of the local system is high reliability; no communication system is required. The disadvantage is that the control parameters that control the local decision can not consider other disaster-related activities. The best system is a combination where the local decision could be overridden from a central location.

Supervisory Control And Data Acquisition, SCADA, systems already exist as part of many water systems. In the event system damage is identified by recognizing excess flows and/or accelerated reservoir drawdowns, operation crews can be dispatched to close valves. The disadvantage of this approach is that it may take too much time (possibly as little as 15 to 30 minutes) to reach the critical valve, before the reservoir is drained. In the Whittier Earthquake, operations crews were able to get to a tank, and close the valve before it drained. In the Loma Prieta Earthquake, response crews were not able to get to critical valves before the tank drained in the San Francisco Auxiliary Water Supply System.

One advantage of a staffed central control system is that the water system will only be shut down as a last resort, only when the reservoir would otherwise drain. It is difficult to have an automated system make such a decision. In the Northridge Earthquake, the Los Angeles Department of Water and Power decided to keep key transmission lines in operation to provide water for fire suppression to a neighboring community, even through they knew reservoirs would be ultimately drained. Major fires were averted.

There is a fine line between "system control" and emergency response based on analysis of hazard information. Systems are in place in both Japan and Southern California where emergency response crews are dispatched based on areas where earthquake damage is expected to be the most severe.

7.3.6 Appropriateness for Different Types of Systems

It is more appropriate to implement a control system on a system that can be shut down without significant consequences. On potable water systems, system shut down may result in water contamination due to back siphonage. Similarly, shut down would render sprinkler systems inoperable. Water from hydrants for fire suppression would be stopped, and water for general use at critical facilities such as hospitals would be stopped. If the situation is critical, system shutdown may still be appropriate; if the system is shutdown inadvertently, or needlessly, there could be significant negative consequences.

Damaged portions of dedicated fire protection systems could be shut down with minimal impact on other users. Isolation systems may have more application on dedicated systems. Construction of a dedicated water system has been economically justified in Vancouver, B.C.. The existing potable water system did not meet fireflow requirements and was in need of a significant pipeline improvement program. The City decided to spend the money on a dedicated system that could be designed to high post-earthquake Performance standards, including a monitoring and control system.

7.3.7 System Hardware

Primary devices for measuring ground motion are available from a number of manufacturers. Instruments with adjustable set points at increasing ground motion levels are available. These devices can be integrated into systems with flow and pressure measuring devices with outputs to controllers that can be programmed to actuate isolation valves under specified conditions. All systems must be designed with its own battery backup system.

Vancouver, British Columbia selected butterfly valves requiring only one-quarter turn by the actuator to be used as the isolation valves in their system. San Francisco uses gate valves that were already in place on their AWSS system. In all cases, a self-contained energy supply for valve actuation must be provided. Compressed air works with one-quarter turn actuators; batteries with motorized operators.

The most economical system can be provided by piggy-backing a seismically activated controller on the pilot control system of an existing pressure reducing valve. These are usually in place between pressure zones cascading down to lower elevations. The advantage of this system is the isolation valve is already in place, and an additional energy source for valve actuation is not required.

7.3.8 Implementation Cost Versus Earthquake Risk

The cost of implementation and system maintenance must be compared against the reduction of earthquake risk. The expense of an installation must be compared against consequences of system failure if it is not installed. Earthquake isolation valves can be more readily justified on major key facilities than on smaller installations.

Areas with a high earthquake risk, such as much of coastal California, can likely justify monitoring and control systems before areas in the mid-west. One water system engineer in the Pacific Northwest related the cost of an earthquake valve to the cost of replacing a significant length of small diameter vulnerable pipe with larger diameter restrained joint ductile iron. He said he would benefit more from the pipe replacement than the isolation valve. The pipe would start working for him the first day it was installed, while the valve may not be needed for one or two generations.

Monitoring and control installations must be maintained. Maintenance of a system for a generation without ever seeing it operate is difficult to justify psychologically as well as being expensive. In more seismically active areas it may be more economically feasible.

The majority of the cost is in the valves and actuator rather than the control system. Use of valves that are already in place for another reason is preferable. A regional earthquake monitoring system is in place in Southern California, and soon to be in place in Northern California. These regional systems may be less expensive for users with numerous facilities.

7.3.9 Monitoring and Control System Reliability

Monitoring and control systems must be reliable. They must function when they are needed, even if that is only once every one to two generations. More importantly, they must not actuate isolation valves inadvertently. There are two examples that focus on the reliability issue. The Seattle Water Department lost there entire telemetry system when a backhoe severed a single cable connecting their control center to the system. This was in a non-earthquake situation.

System reliability can be increased by providing system redundancy in both the reservoir and pipeline system as well as in the control system. The system should also be designed to remain functional following the design earthquake.

7.3.10 Northridge and Kobe Earthquake Water System Performance

The Los Angeles Department of Water and Power, LADWP, did not have any earthquake valves in their water system during the Northridge Earthquake. They suffered approximately 1,200 pipeline failures, and lost service to much of the San Fernando Valley. They could not provide water from the system for fire suppression in all areas. LADWP staff made decisions not to close valves to isolate pipelines in order to maintain service for fire suppression. It is unknown whether a monitoring and control system would have improved system performance.

The Kobe Water Department had a monitoring and control system in place on 21 reservoirs at the time of the Kobe Earthquake. The system worked on 18 reservoirs. Each of the reservoirs was one of a pair; the other reservoir remained online. The objective of the system was to isolate three liters of drinking water per person to last for seven days following the earthquake. Even though it functioned, there were many complaints about inadequate drinking water.

7.3.11 Conclusions and Recommendations

Monitoring and control systems offer an alternative to mitigate water system earthquake damage at a moderate cost compared to pipeline replacement. The effectiveness of monitoring and control systems should be considered on a case by case basis. More seismically active areas are more likely to benefit. System reliability is crucial.

7.4 MITIGATION ALTERNATIVES: AUXILIARY SUPPLIES

Supplemental or auxiliary water supplies should be identified by fire and water agencies that can be utilized for emergencies. Generally, this would include all supplies of 10,000 gallons or greater. Swimming pools, storage tanks, ponds, cisterns, reflecting pools, lakes, and all other manner of natural water sources should be identified, together with the means of access to such supplies. This information should be collated into operations centers of fire and water departments into computers, manuals and emergency directories. A plan should be developed as to how these supplies can be used in emergencies. Back flow prevention must be addressed in use of all auxiliary supplies so that drinking water systems do not become polluted during fire fighting operations.

7.4.1 Auxiliary Supplies

Suction connections are one means for fire departments to have pre-installed connections to static supplies such as bays, lakes, reservoirs, tanks, and rivers. Suction connections allow access problems for fire department pumpers to such supplies to be minimized. Often it is difficult to access static water supplies because of the nature of the geography and construction of buildings, walkways and piers that prevent positioning of a pumper truck for drafting access.

Suction connections, which consist of a six inch pipe into the water, (below low tide or low water marks), with a suction connection at the street level so a pumper truck can connect its hard suction for operations. The City of San Francisco has recently installed 42 such connections along its waterfront for fire department use. The Marin Municipal Water District in Marin County, California, recently installed special fire department connections to many tanks to allow supply to be taken even if the water mains from such tanks were not in service for any reason.

A more recent development is the use of large diameter hose, (5, 6, 8 and 12 inch), as a means of establishing emergency above ground distribution systems or feeders in time of emergency. Some cities have adopted specialized apparatus and equipment to utilize this new capability. This type of equipment and systems must be coordinated into the overall emergency plan of the fire and water departments.

Use of auxiliary supplies together with above ground distribution system capability takes considerable planning, training and coordination and between fire and water agencies. In areas where earthquake hazard and wild land hazard is significant such planning is vital for successful emergency operations when disaster events occur. Fire Departments are generally more prepared for emergency operations as they do this as a matter of routine on a daily basis. Water Departments are more geared to the planning and developmental process, with long term main replacement programs in use and daily service repairs being accomplished. The goals of the fire and water agencies can be to utilize the best capabilities of each in developing the emergency plans, equipment and most efficient use of available facilities.

7.5 USE OF GIS TO EVALUATE EARTHQUAKE HAZARD EFFECTS AND MITIGATION ON PIPELINE SYSTEMS

7.5.1 Introduction

Geographic information systems, GIS, were developed in the 1980s in parallel with mini (UNIX based) (ESRI, 1995) and micro (personal) (MapInfo, 1996) computers. During this same time frame, the lifeline earthquake engineering technical community began to develop a better understanding of earthquake impacts on pipeline systems.

In the early 1990s, these three technologies, computers, GIS, and lifeline modeling, came together allowing end users to employ them at a reasonable cost. All of a sudden, many communities were putting geologic hazard data and pipeline data in GIS format, along with many other types of electronic information. Historically, the most significant cost associated with conducting a computer modeling study was collecting the data. Now, in many cases, the data was already available in electronic format, waiting to be manipulated to achieve results useful to the end user. In the cases described herein, the end users are water and sewer service providers.

Now, in the United States, use of GIS to evaluate pipeline vulnerability is becoming a standard practice. The procedure is generally described in a publication of the American Water Works Association (Ballantyne, 1994).

This section describes earthquake pipeline loss modeling procedures and their application in several communities. The use of pipeline loss results is described to give the reader an opportunity to identify opportunities for application.

7.5.2 Use of Results

Typically, pipeline damage is estimated in terms of number of failures per unit length and segment of pipeline. This information can then be used to estimate the functionality of a water or sewer system for a given earthquake scenario. If the system is not expected to be functional, the information can be used to estimate the time it will take to restore the system to full operation. There is a wide variety of applications for this information, with new uses continuing to be identified regularly.

7.5.2.1 Emergency Preparedness

One of the crucial elements of an earthquake mitigation program is emergency planning. Immediately following an earthquake, resources required for effective response are in limited supply, and time becomes a very important parameter. GIS can be used to optimize use of these resources. Several examples of uses along with places where they were used are described below.

- Develop emergency operation strategy - in Bellevue, Washington, results of the water system pipeline evaluation (Dames & Moore, 1996) were used to identify locations where earthquake valves could be effectively employed in the system to isolate vulnerable pipelines.

- Estimate resources required for restoration such as staff, equipment, repair materials. In Seattle, Washington, Seattle Metro used pipeline loss estimates to develop an inventory of repair materials required for post-earthquake recovery.

7.5.2.2 Mitigation

GIS is being used as a tool to optimize mitigation planning. As utility budgets continue to shrink, it is crucial to make the best use of available funds for both capital improvements and maintenance. Some applications are described below:

- Compare estimated system performance with planning objectives - at the Portland Bureau of Environmental Services, policy makers set post-earthquake sewer system performance objectives. GIS was used to estimate expected system performance for several levels of earthquakes for comparison against the objectives to establish capital improvement requirements.

- Identify, and prioritize upgrade for deficient pipeline segments/areas/functions - At the Marin Municipal Water District, just north of San Francisco's Golden Gate Bridge, GIS results (Kennedy/Jenks, 1995) were used to identify pipeline segments that were vulnerable, and crucial to maintain system operation, and combining these parameters to prioritize mitigation efforts.

7.5.2.3 Dollar Losses

GIS systems can quickly calculate dollar losses once units of damage have been estimated, and has a tremendous ability to put the information in dazzling graphic format. Applications include:

- Estimate probable maximum losses to use as a tool in negotiating insurance coverage and premiums - The Unified Sewerage Agency in Hillsboro, Oregon, used GIS results (Dames & Moore, 1995) to better understand their exposure to earthquake damage to pipelines for discussions with their insurance provider.

- Influence decision makers - loss estimates, including pipeline losses are compelling information to influence decisions makers to fund future earthquake programs. The color graphics generated by GIS are excellent presentation graphics for key meetings. The GIS maps for the Marin project are key components of the package being assembled to provide voters information on an $80 million bond issue for water system improvements.

- Quickly calculate dollar losses for governmental agencies to speed assistance - EPEDAT was used by EQE to provide early damage losses to the California Office of Emergency Services to negotiate funding from the Federal Emergency Management Agency following the Northridge Earthquake.

7.5.2.4 Corridor Selection for New Pipelines

For new systems, GIS is an excellent tool to assess relative risk, both earthquake and non-earthquake, for different pipeline corridors.

7.5.3 Procedure

GIS offers a framework to conduct pipeline loss studies. This section steps through the pipeline loss modeling process.

7.5.3.1 Seismicity

Typically, earthquake scenarios are selected that represent standardized probabilities of exceedance, or otherwise meaningful to the particular community. Often, operating basis earthquakes are selected that have a 50% chance of exceedance in a 50 year life of a facility. The premise is that the facility will probably experience the defined intensity of shaking during its life, and should be designed to remain functional. The design basis earthquake, DBE, is often taken as an intensity that has a 10% chance of exceedance in that same 50 year life of the facility. The facility will probably not be subjected to the prescribed earthquake intensity during its life, but may, and should be designed at least to maintain life safety. The Uniform Building code used in the Western United States uses this standard.

It is much easier to model an earthquake scenario rather than a probabilistic earthquake ground motion. Usually, a scenario is selected that represents the ground motions comparable to the OBE or DBE.

In some cases a specific earthquake scenario is of greater interest. In the Marin Municipal Water District project that is described herein, an event on the near by San Andreas Fault was one selected scenario because it will likely occur within the next 100 years, and heavily impact the District.

Once these earthquake scenarios are selected, ground motion intensities are calculated across the study area using the GIS.

7.5.3.2 Hazard Mapping

Geologic hazards are mapped in electronic format. To the extent possible, existing hazard mapping information is obtained. Washington, Oregon, and California all have hazard mapping programs with maps available for many urban areas. The USGS has also funded a significant hazard mapping effort. There is a wide variation of the quality of information available.

Typically liquefaction and landslide susceptibility are the basic hazard maps required. In California, surface faulting must be taken into account. In Washington and Oregon, surface faulting is unusual, and therefore not considered. In more sophisticated studies, site amplification is taken into account to better establish liquefaction and landslide probabilities (calculated for the specific scenario shaking intensity). Site amplification may also influence damage from wave propagation effects, although these are considered to be small compared to permanent ground displacement effects.

Key parameters required to achieve reasonable certainty of results include areal extent of liquefaction and landslide, and lateral spread displacement. In earlier studies, it became apparent that an estimate of the percentage of the area that would deform as a result of liquefaction was different, and probably much less than liquefaction probability. Geologists started to provide an estimate of areal extent. Lateral spread displacements are an important relationship in establishing the segmented pipe failure rates. Youd and his colleagues provided the LSI and MRL techniques to estimate displacements (Youd & Perkins, 1987; Bartlett & Youd, 1992). GIS provides an excellent tool to calculate displacements using digital elevation maps to calculate slopes and identify free faces used in the MRL calculation.

Refer to Table 7-5 for an example of how the liquefaction hazard parameters are developed and input into the GIS for the Marin Municipal Water District project.

Visualization of these parameters using GIS is very useful in gaining an understanding of the regional geologic hazard environment.

7.5.3.3 Pipeline Damage Algorithms

Pipeline damage is estimated by applying damage algorithms relating earthquake geologic hazards to pipeline categories, each with different parameters. These pipeline categories are then mapped on GIS. Pipeline parameters that are usually taken into account include:

- Ductility
- Geometry
- Condition
- Joint flexibility
- Joint restraint

Basic algorithms to estimate wave propagation losses relate Modified Mercalli Intensity, MMI, or Peak Ground Velocity, PGV, to pipeline damage. Most of the empirical damage data used to develop these algorithms is based on MMI. An effort is underway to use PGV as the standard ground motion parameter for pipeline wave passage damage as it is thought to be more representative of wave propagation effects.

Early damage algorithms for permanent ground displacements related liquefaction or landslide susceptibility to pipeline damage. The use of GIS has allowed easier input of parameters required to estimate ground displacements. Recent damage algorithms relate permanent ground displacement to pipe damage. This relationship is most applicable to segmented pipe. Continuous pipe damage is more closely related to lateral spread block size (O'Rourke, 1992). The pipeline community is interested in obtaining information on hazard mapping techniques establishing block size.

The results of the pipeline analysis are in terms of pipeline failures per unit length and per segment (node to node).

Results are presented on graphs or tables in standard categories that are based on the probability of damage in 1 km of pipe as shown in Table 7-6. Graphical presentation makes vulnerable areas become very apparent.

7.5.3.4 Dollar Losses and Restoration Time

Once the number of pipe failures is calculated, repair costs can be directly calculated. Restoration time can be estimated as a function of the number of failures, available resources and repair crews.

7.5.3.5 Hydraulic Analysis

Once the number of pipeline failures is known, a system hydraulic analysis can be performed using a standard computer network hydraulic analysis package. A rule of thumb is that 20% of the failures are breaks that result in loss of pipeline hydraulic continuity. An exception to this rule is in areas of fault rupture where a much higher percentage of failures are likely to be breaks. These systems have been refined to account for negative pressures.

7.5.3.6 Pipeline Criticality

If the objective is to prioritize pipeline deficiencies for a mitigation program, pipeline criticality is developed. Pipeline criticality is a function of the pipeline redundancy, capacity, and/or ease of repair. In the Marin Municipal Water District study, pipeline criticality ranged from 5 (high) to 1 (low).

In Marin County, the single raw water pipeline from the source to the treatment plant, and from the treatment plant to the distribution system, was given the highest criticality rating of 5. In another instance in Marin county, two redundant pipelines carried water between two points. Both were highly vulnerable. One was inaccessible. The other accessible pipeline was given a high criticality rating because it is the one the District would depend on, repairing it if necessary, following an earthquake.

7.5.3.7 Prioritization of Deficiencies and Mitigation

Pipeline mitigation is prioritized considering both vulnerability and criticality, and is derived from a matrix as shown in Table 7-3.

7.5.4 Conclusions

1. GIS and computer hardware development are making powerful GIS systems readily available to end users.

2. Estimating earthquake losses to pipeline systems using GIS is becoming common in areas of high and moderate seismicity in the United States.

3. Estimates of earthquake damage to water and sewer systems is useful for emergency preparedness, mitigation optimization, making dollar loss estimates, and corridor studies for new pipelines.

4. Current procedures quantify earthquake hazards, pipe parameters, and component criticality leading to prioritized mitigation recommendations.

5. We should continue to gather information relating permanent ground displacements and ground shaking intensity to pipeline damage.

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TABLE 7-1 SEISMIC VULNERABILITY RATINGS (SVRs)

Seismic Vulnerability	Seismic Performance Estimate	SVR
Severe global deficiencies	Partial to total collapse likely Life threatening situation (moderate to high probability) Structure is condemned/unrepairable Needs substantial strengthening work	5
Major global deficiencies	Substantial damage to partial collapse Life threatening situation (moderate probability) Damaged but operable within 21 days Needs major strengthening work	4
Major localized deficiencies	Major damage Life threatening situation (low probability) Damaged but operable within 24 hours Needs moderate strengthening work	3
Minor localized deficiencies	Minor damage No life threatening concerns Damaged but operable Needs minor strengthening work	2
No apparent deficiencies	Little or no damage No life threatening concerns Little or no function disruption Structure is adequate, no work needed	1

TABLE 7-2 DAMAGE RATE AND ASSOCIATED SVR

Vulnerability Class	Damage Rate (damages/km)	SVR
High	5.0 Damage Rate	5
Medium High	1.0 Damage Rate < 5.0	4
Medium	0.1 Damage Rate < 1.0	3
Medium Low	0.01 Damage Rate < 0.1	2
Low	Damage Rate < 0.01	1

TABLE 7-3 RISK ANALYSIS MATRIX SHOWING SEISMIC RETROFIT PRIORITIES A, B, C, AND D (A=HIGHEST PRIORITY)

Overall Criticality Rating (5 = Most Critical)	Seismic Vulnerability Rating As Measured by Seismic Performance Estimate (5 = Most Vulnerable)				
	>4.5	3.6 - 4.5	2.6 - 3.5	1.6 - 2.5	<1.6
>4.5	A	A	B	C	Acceptable
3.6 - 4.5	A	B	C	D	Acceptable
2.6 - 3.5	B	C	D	Acceptable	Acceptable
1.6 - 2.5	C	D	Acceptable	Acceptable	Acceptable
<1.6	Acceptable	Acceptable	Acceptable	Acceptable	Acceptable

TABLE 7-4 EXISTING LIFELINE SYSTEM MONITORING EXAMPLES

Seismic Hazard Parameters	Structural Component/ Soil Parameters.	System Operational Parameters
Water Reservoirs - earth-quake valves close on PGA threshold exceedance	Water Pipe, Joint Displacement - strain gages	Water Systems - control system on pressure, flow, rate of change of flow, or mass balance between points
Transmission system and individual service shutdown on PGA threshold exceedance	Tokyo Water - tunnel monitored with strain gages	Power, Communications-control system based on voltage, load
Electric Power-shut down services to control fire ignition source on re-energizing.	Tokyo Gas - in-situ liquefaction monitoring	Natural Gas, Oil Transmission Systems - shutdown on excess flow or pressure reduction.
Trains - Us & Japan - shutdown power on PGA threshold exceedance.	Building, Bridge Structures - strain gages	Transportation - monitor traffic with video camera allowing quick dispatch of emergency equipment

TABLE 7-5 LIQUEFACTION SUSCEPTIBILITY CATEGORIES

Category	Susceptibility	Estimated Displacement (Meters)	Fraction of Area Estimated to Liquefy (Areal Extent)	Estimated Trigger Level (Fraction of Gravity)
1	Very Low	0	0	Not Applicable
2	Low	0.0-0.07	0.01	0.5
3	Moderate	0.0-0.10	0.02	0.3-0.5
4	High	0.05-0.40	0.1	0.1-0.3
5	High with Free Face	0.05-1.0	0.2	0.1-0.3
6	Very High	0.05-1.0	0.25	0.1
7	Very High with Free Face	0.01-2.5	0.5	0.1

TABLE 7-6 PIPELINE VULNERABILITY CLASS

Vulnerability Class	Damage Rate	Probability of Damage in 1 km	Vulnerability Rating
High	1.4< Damage Rate	0.75< probability	5
Moderate - High	0.7<Damage Rate<1.4	0.50<prob.<0.75	4
Moderate	0.1<Damage Rate< 0.7	0.10<prob<0.50	3
Low – Moderate	0.01< Damage Rate < 0.1	0.01<prob<0.10	2
Low	Damage Rate < 0.01	probability<0.01	1

8.0 CONCLUSIONS AND RECOMMENDATIONS

8.1 STUDY CONCLUSIONS

From this study we can draw six conclusions summarized below. Understanding these conclusions will assist water and fire departments in developing mitigation programs to enhance post-earthquake water system performance.

8.1.1 Post-Earthquake Functionality

Immediate post-earthquake water system function is crucial, particularly to suppress fires. This has been demonstrated in earthquakes in Kobe, Japan, 1995; Northridge, California, 1994; Loma Prieta, California, 1989; Kanto, Japan, 1923; and San Francisco, California, 1906. In every case, water systems failed, and impaired fire suppression.

8.1.2 Water and Fire Department Common Understanding and Communication

Water system vulnerability must be understood by both water and fire departments. Water system owners then have the opportunity to improve the system hardware before the earthquake to reduce system vulnerability. They can also develop emergency response programs to enhance post-earthquake system performance. Fire departments can work with water system owners to promote system mitigation. Fire departments can identify alternative water supplies and develop methods to effectively use those supplies in the event the potable supply fails.

Communication between fire and water departments becomes critical following earthquakes. Water system performance may be compromised. Fire departments must clearly communicate to water departments where , and how much water for fire suppression is needed. Water departments must clearly communicate with fire departments if, when, and how much water is available at those locations.

8.1.3 Pipeline Damage Causes System Failure

Transmission and distribution pipeline damage has been the key element causing water systems to fail in every earthquake summarized in this report. The common scenario is that numerous pipelines fail, and water is drained from the system within minutes or hours of the earthquake.

Pipelines subject to permanent ground deformation caused by liquefaction, landslide, and fault offset are the most significant pipeline hazards. Pipelines constructed of brittle materials with rigid joints are the most likely to fail.

8.1.4 Definition of Post-Earthquake Performance Criteria

An important initial step in developing a mitigation program is to define post earthquake performance criteria. Performance criteria should define desired, or acceptable performance. These criteria can then be used to guide development of an earthquake mitigation program.

8.1.5 Water System Reliability Methods

Water system reliability can be assessed using deterministic, probabilistic, fault tree, and/or GIS techniques. These techniques provide varying levels of detail in the assessment with associated varying levels of effort required for implementation. The most widely used technique has been to use a deterministic approach for defined earthquake scenarios. The earthquake scenarios are typically selected to represent events that are likely to impact the region. The earthquake magnitude and location of scenarios are often selected for "standardized" probabilities of exceedance, such as 50 percent and 10 percent in 50 years.

Probabilistic and fault tree techniques have been used extensively in the nuclear industry, but have found limited application in water system analyses.

8.1.6 Detailed Evaluation Techniques

Detailed evaluation methods are available for pipelines, reservoirs, and other system components. Evaluation techniques using empirical data are the most widely used for segmented pipe subjected to earthquake wave propagation. Structural analysis techniques considering soil-pipe interaction have been developed for pipelines subjected to permanent ground deformation.

The American Water Works Association tank design standards address seismic design of new steel and post-tensioned concrete tanks. These same methods can be applied to existing tanks. A less conservative approach based on use of empirical data has been developed for application to at-grade steel tanks.

There is no single source of information applicable to seismic evaluation of all water system facilities. The best reference is the AWWA document, *Minimizing Earthquake Damage, A Guide for Water Utilities* (Ballantyne, 1994).

8.2 RECOMMENDED MITIGATION STRATEGIES

Considering the six conclusions discussed above, the five strategies described below are recommended to mitigate the effects of earthquakes. Application of these strategies will enhance the post-earthquake performance of water systems to provide water for fire suppression and potable use.

8.2.1 Fire and Water Department Education and Communication

Improve the fire and water departments' understanding of the earthquake vulnerability of water systems. This can be accomplished through continuing education programs and participation in industry sponsored workshops. In addition, each water system is different, and must be assessed on an individual basis to gain an understanding of its vulnerabilities.

Enhance communication between fire and water departments. Cities with water and fire departments under their jurisdiction should asses inter-department communication and, depending on the findings, develop policy and procedural changes to reduce any institutional barriers. Water and fire organizations under different jurisdictions should aggressively seek out one another, and develop coordinated efforts to address earthquake mitigation and response concerns.

8.2.2 System Hardware

Upgrade system hardware, prioritized considering the criticality and vulnerability of each component. Many system components can be effectively upgraded at low to moderate costs. Component upgrades can be integrated into other capital improvement or maintenance projects.

Hardware mitigation programs should be prioritized considering the criticality or importance of the component within the system, the vulnerability of the component to earthquakes, and the cost and associated benefit of the upgrade.

8.2.3 Monitoring and Control Systems

Provide post-earthquake system monitoring and control to allow rapid isolation of damaged sections of the system. Pipeline systems are very expensive to replace. Sometimes, even if they are replaced with "modern" pipeline materials, they may still be damaged in earthquakes. Implementation of a monitoring and control system to automatically isolate storage and/or damaged portions of the system can be an effective technique to control earthquake damage consequences.

8.2.4 Alternatives Supplies/Sources

Develop alternative sources/supplies of water for fire suppression. Providing operational flexibility may be the most effective approach to improving service reliability. Many communities have water supplies already in place that can be tapped during emergencies, particularly for water for fire suppression. Such alternative sources/systems can include planned/developed use of bays, lakes, rivers, drainage channels, or swimming pools within the community. These can be accessed using dedicated fire protection systems, portable fire protection systems, permanent drafting stations, fire boats, and floating pumps.

8.2.5 GIS

Use GIS. It is a powerful tool, effective in performing earthquake mitigation programs, and responding to and recovering from earthquake events.

GIS provides the capability to organize, manipulate, and graphically present information used in earthquake mitigation programs. Hazard and pipeline data can be gathered and manipulated. The vulnerability of pipelines can be calculated considering geotechnical and pipeline mechanical parameters available through the GIS. Recommended mitigation programs can be effectively portrayed for presentations to decision makers.

APPENDIX A

Appendix A consists of papers documenting water system performance during eight moderate to large urban earthquakes, plus the 1993 Des Moines, Iowa flood, and the 1991 Oakland Hills, California firestorm. The authors of these papers are listed below the titles of the papers. Only minor edits were made by the Principal Investigators where necessary.

A.1a 1989 LOMA PRIETA, CALIFORNIA EARTHQUAKE: WATER SYSTEM PIPELINE DAMAGE

By J. Eidinger

The Loma Prieta earthquake (magnitude 7.1) of October 17, 1989 at approximately 5:00 PM, caused varying levels of damage to the water systems within a 70 mile radius of the epicenter.

A.1.1 Pipeline Damage

Pipeline damage was caused by four geotechnical hazards, listed in order of their relative importance: liquefaction, ground shaking, landslides, and surface faulting. Ground shaking refers to the transient soil deformations due to seismic wave propagation. Pipeline damage in areas not subject to landslides, liquefaction or surface faulting is attributed to ground shaking. Landslides are the permanent displacement of soil mass which can be very damaging to buried pipe. Liquefaction is a phenomenon that occurs in loose, saturated, granular soils when subjected to long duration, and strong ground shaking. Localized permanent ground displacements occur in surface fault rupture areas.

Due to the spatial variations in the level and extent of geotechnical hazards, the level of pipeline damage seen in the Loma Prieta earthquake was not uniform. Some of the most significant pipeline damage occurred over 40 miles from the epicenter, where the combination of built-up environment on poor liquefiable soils was not uncommon. In the following paragraphs, we summarize the observed pipeline damage, beginning with the epicentral region, and then for locations further away.

Nearest the epicentral region, ground shaking accelerations exceeded 0.5g in many areas, with attendant ground velocities over 25 inches per second. The epicentral region was located in sparsely populated mountainous terrain, with relatively little inventory of buried pipelines. Landslides were common, but fortunately affected few buried pipelines due to the remoteness of the region.
Just outside the epicentral region, ground shaking was still intense, on the order of 0.35g to 0.5g. This level of shaking affected the community of Santa Cruz (population 45,000). There were about 240 pipeline main leaks / breaks in the Santa Cruz area. Most of the damage was to cast iron pipes with leaded joints; little damage was reported to rubber gasketed ductile iron and asbestos cement pipe. There were over 250 service line problems. The overall pipeline repair rate was on the order of 1.5 to 2.0 repairs per mile of buried pipeline.

Outside the epicentral area were the communities of Watsonville (pop. 25,000), and Hollister (pop. 12,000). Ground motions in these areas were less intense than in the epicentral area, but were still on the order of 0.3g - 0.4g (Watsonville) to 0.25g - 0.3g (Hollister). There were about 60 pipeline main leaks / breaks in these and adjacent areas, plus an additional 70 service line leaks. The overall pipeline repair rate (excluding service lines) was on the order of 0.5 repairs per mile of buried pipeline.

To the north of the epicentral area is the major city of San Jose, with surrounding communities of Cupertino, Campbell, Los Gatos and Los Altos (total population 900,000). Much of the area lies in an alluvial plain, with relatively little built environment in soils strongly susceptible to liquefaction. Ground motions in these areas were on the order of 0.2g - 0.25g. There were about 120 pipeline main leaks / breaks and 10 service line leaks in these areas. The overall pipeline repair rate (excluding service lines) was on the order of 0.05 repairs per mile of buried pipeline.

The other two water systems strongly affected by the Loma Prieta earthquake were the City of San Francisco (population 750,000) and the East Bay Municipal Utility District (population 1,200,000). The San Francisco water system was about 35 miles north of the epicentral region. The EBMUD system varies between 35 and 70 miles northeast of the epicentral region. Ground motions for most of San Francisco were about 0.10g. There were about 150 pipeline main leaks / breaks in the San Francisco water system, of which about two-thirds were concentrated in the Marina District (subjected to amplified motions and some soil settlements); plus an additional 100 service line leaks. The overall pipeline repair rate (excluding service lines) was on the order of 0.2 repairs per mile of buried pipeline. Ground motions for most of The EBMUD system were about 0.10g. There were 113 pipeline main leaks / breaks and 22 service line repairs (on the utility side of the meter). About half the damage was concentrated in areas with poor soil conditions, where local ground motions were about 0.20g, and where there was a highly corrosive environment; in a very few locations, damage was attributed to settlements ranging from 1 inch to several inches. The remaining damage occurred in firm alluvial locations without liquefaction. The overall pipeline repair rate (excluding service lines) was on the order of 0.03 repairs per mile of buried pipeline.

Figure A-1.1 correlates pipeline repair rate with generalized peak ground velocities for the five water service areas described above. The data point second from the left (San Francisco) is relatively high due to the high incidence of liquefaction for that water system.

A.1.2 Analysis of Pipeline Damage

Figure A-1.1 shows a definite trend in repair rate with input ground motion level, as measured by surface level peak ground velocity (PGV). However, there is scatter which cannot be explained solely by PGV. Pipeline damage was also affected by local area permanent ground deformations, PGDs; as well by pipeline material, age, corrosion and diameter.

A.1.2.1 Pipeline Material.

An analysis of pipe repairs by pipeline material was conducted using the EBMUD pipeline database. Table A-1.1 shows the results, not differentiating for levels of ground shaking.

TABLE A-1.1 Repairs to EBMUD Water Distribution System Pipes

Material	Repairs	Repair Rate (Per 1,000 Feet of Pipe)
Cast Iron	52	0.007
Steel	46	0.012
Asbestos Cement	13	0.002
PVC	2	0.002
Service Connections	22	
Total	135	

The above repair rate for steel pipe is higher than that for other pipeline materials. This high repair rate occurs because EBMUD intentionally uses only steel pipe in poor (highly corrosive, prone to settlement) soil areas, as part of the normal design process. The good performance of AC and PVC pipe was also reported at other sites close to the Loma Prieta epicenter. This was attributed to their rubber gasketed articulated joints which provide flexibility to withstand modest seismic ground movements (Pickett et al, 1991). Figure A-1.2 correlates repair rates versus input motions, for different pipeline materials.

A.1.2.2 Age

The Loma Prieta earthquake showed that older pipes had a higher incidence of failure than newer pipes. Figure A-1.3 covers about 1,000 miles of lap arc-welded steel pipe. Pipe damage due to the 1987 Whittier Narrows Earthquake (Los Angeles area) clearly showed a trend of earthquake pipe breaks vs. age of pipe (Wang, 1990). Note that age effects are likely strongly correlated with corrosion effects, due to the increasing levels of corrosion over time.

A.1.2.3 Corrosion

Corrosion weakens pipe due to the effective decrease in material thickness as well as creating stress concentrations. Screwed / threaded steel pipes appear to fail at a higher rate than other units of steel pipes. Some cast iron pipes have also experienced higher incidences of corrosion failure (Isenberg, 1978, 1979; Isenberg and Taylor, 1984).

Figure A-1.4 shows the repair rates for cast iron (inventory of about 1,000 miles) and steel pipe (inventory of about 1,000 miles) according to the relative soil corrosiveness. In areas where corrosion is not too important (low-corrosive and medium-corrosive soils), segmented cast iron pipe fared worse than welded steel pipe, as expected. In areas where corrosion is important (high-corrosive soils), welded steel pipe fared worse than segmented cast iron pipe.

A.1.2.4 Diameter / Appurtenances and Branches

Experience has shown that pipeline damage tends to concentrate at discontinuities such as pipe elbows, tees, in-line valves, reaction blocks, and service connections. Such features create anchor points (rigid locations) that will promote force/stress concentrations. Locally high stresses can also occur at pipeline connections to adjacent structures (e.g., tanks, buildings and bridges), especially if there is insufficient flexibility to accommodate relative displacements between pipe and structure.

This issue also relates to pipeline diameter. Figure A-1.5 (inventory of 1,000 miles of welded steel pipe) shows that larger diameter pipelines (12" and larger) usually have much fewer repair rates due to fewer appurtenances and branch connections than for smaller diameter pipelines. This is because large diameter pipes have few, if any, service line connections or hydrant lateral connections. This was observed in the Loma Prieta earthquake.

A.1.3 Pipeline Damage Algorithms - Ground Shaking

We developed new buried pipeline fragility data that correlate pipeline repair rates to input PGV or PGD. The fragility curves are further described in Eidinger et al, (1995). The following paragraphs highlight the methodology.

Data from the 1989 Loma Prieta earthquake is combined with data from the 1965 Puget Sound, 1969 Santa Rosa, 1971 San Fernando, 1983 Coalinga, 1985 Mexico City, and 1989 Tlahuac Mexico earthquakes. The best fit fragility curve, $n = 0.00032 * (PGV)^{1.98}$, should be multiplied by constant K_1 (Table A-1.2 below) to adjust this curve for various pipe materials (i.e., total repair rate = $n * K_1$, in repairs per 1,000 feet of pipe). The "quality" column in the table describes the confidence in the current empirical data (quantity and completeness) to support the relative K_1 factors. The K_1 factors have at best one significant digit. In suggesting the "quality" of the data, "B" suggests that there is a reasonable amount of backup empirical data and study; "C" suggests that there is limited empirical data and study; "D" is based largely on extrapolation and judgment, with very limited empirical data. Based on the recent 1995 Kobe earthquake, the K_1 factor for ductile iron pipe with rubber gasket joints appears to be in the range of 0.5. The lack of "A" ratings suggests that there is still much room for future improvement in our current understanding of buried pipeline earthquake fragility.

TABLE A-1.2 - Fragility Curve Parameter, K_1

Pipe Material	Joint Type	Soils	Diam.	K_1	Quality
Cast iron	Cement	Unknown	Small	0.8	B
Cast iron	Cement	Corrosive	Small	1.1	C
Cast iron	Cement	Non corr.	Small	0.5	B
Cast iron	Rubber gasket	Unknown	Small	0.5	D
Welded steel	Arc welded lap	Unknown	Small	0.5	C
Welded steel	Arc welded lap	Corrosive	Small	0.8	D
Welded steel	Arc welded lap	Non corr.	Small	0.3	B
Welded steel	Arc welded lap	All	Large	0.15	B
Welded steel	Rubber gasket	Unknown	Small	0.7	D
Asbestos cement	Rubber gasket	All	Small	0.5	C
Asbestos cement	Cement	All	Small	1.0	B
Asbestos cement	Cement	All	Large	2.0	D
Concrete	Welded	All	Large	1.0	D
Concrete	Cement	All	Large	2.0	D
PVC	Rubber gasket	All	Small	0.5	C
Ductile iron	Rubber gasket	Non corr.	All	0.3	C

For pipe having diameters of 12 inches or greater, the repair rates are adjusted such that 1/2 of the damage are breaks and the rest leaks. This considers that large diameter pipes have fewer service connections, and are less prone to corrosion than small diameter pipes.

A.1.4 Pipeline Damage Algorithms - Permanent Ground Deformations

The seismic hazard is defined by the amount of PGD caused by either soil liquefaction or landslide. The repair rate is dependent on the magnitude of PGD the particular pipe segment experiences.

The corresponding fragility curve for estimating pipe repairs due to PGDs is as follows:

$n = 1.03 * (PGD)^{0.53}$ (n = repair rate per 1,000 feet of pipe, PGD in inches). As with ground shaking, this fragility formulation should be multiplied by a constant K_2, to account for pipe materials and joinery (Table A-1.3). Most empirical damage data has been for pipes located in poor soil areas, and therefore a soil corrosivity index is implied in the empirical data. With the exception of cast iron pipe (mostly smaller diameter), the empirical evidence is very limited, as denoted by the Quality index. The recent 1995 Kobe earthquake showed that for ductile iron pipes fitted with special slip joints capable of 2 - 3 inches of movement at every joint did quite well in moderate PGDs (K_2 less than 0.1); more study of ductile iron and special slip joint performance is clearly mandated.

TABLE A-1.3 - Fragility Curve Parameter, K_2

Pipe Material	Joint Type	K_2	Quality
Cast iron	Cement	1.0	B
Cast iron	Rubber gasket, mechanical	0.7	C
Welded steel	Arc welded lap	0.15	C
Welded steel	Rubber gasket	0.7	D
Asbestos cement	Rubber gasket	0.8	C
Asbestos cement	Cement	1.0	C
Concrete	Welded	0.8	D
Concrete	Cement	1.0	D
PVC	Rubber gasket	0.8	C
Ductile iron	Rubber gasket	0.3	C

A.1.5 Storage Tanks

The Loma Prieta earthquake showed that above ground storage tanks are vulnerable to damage. The major damage modes observed, for locations outside of liquefaction areas, were: sliding of unanchored tanks, especially at high g levels near the epicentral area; breakage of inlet-outlet lines, due to uplift of unanchored tanks (moderate g levels); failure of prestressed concrete tanks due to hoop overstress; and, damage to roofs due to slosh load impacts. The following paragraphs summarize the damage for specific tanks.

A 1,000,000 gallon welded steel tank, built in 1971, buckled at the shell-roof connection in Watsonville (PGA likely over 0.35g). The tank continued to hold water and function. Nine other tanks in this area had no damage.

A 750,000 gallon at-grade unanchored welded steel water tank at Moss Landing cracked at the bottom plate, leaking all its water. Inlet-outlet pipes for two other similar tanks were also damaged. (PGA likely over 0.35g)

In Scotts Valley, unanchored 750,000 gallon and 400,000 gallon welded steel tanks on concrete ring walls had damage at their wood roof to steel shell connections. Two tanks drained due to breakage of side-located inlet-outlet pipes. There was no damage to one other 1,250,000 gallon unanchored welded steel tank, constructed in 1983 per AWWA code, located on a concrete ring foundation atop bedrock. (PGA likely over 0.35g).

The inlet-outlet pipe beneath a 700,000 gallon welded steel tank separated from the floor plate (uncertain if tank uplift caused this) (PGA likely over 0.30g).

A 10,000 gallon redwood tank collapsed (PGA likely over 0.30g).

A 600,000 gallon unanchored welded steel tank in Pajaro performed well (designed per AWWA 1986 code). (PGA likely over 0.25g).

A 100,000 gallon unanchored bolted steel tank developed an elephant foot buckle (PGA likely over 0.30g).

A 1,100,000 gallon wirestressed concrete tank failed in the Los Altos hills (PGA over 0.18 g), suddenly releasing its entire water contents. The failure mode was a vertical 4-inch wide crack, the height of the tank. The hoop direction wires failed.

A 200,000 gallon welded steel tank in Sunny Mesa, constructed in 1968, tilted and broke its side-located inlet-outlet pipe. The tank itself tilted 2" after the earthquake, although it could still hold water (with the exception of the broken inlet outlet pipe).

In the San Lorenzo Valley, five redwood tanks were lost, ranging from 10,000 gallons to 150,000 gallons.

In Hayward, there was modest damage to a 50,000,000 gallon open cut lined reservoir, built in 1956. A precast concrete roof system covers the reservoir. Interior inspections in 1992 revealed that about 1% to 2% of the concrete panel to column connections had been dislodged. In a few locations, steel bars were exposed. From the outside, damage to the roof would not be observed. The site experienced ground motions in the range of 0.10g to 0.15g. The original design basis for the roof was likely an equivalent static load of 0.10 times the weight of the roof. The damage is considered severe enough to preclude people from walking on the roof; however, the reservoir remains in service through 1996. Currently, the water utility has conceptual plans to retrofit the roof; but the high cost and perceived modest benefit of such retrofit may preclude implementation.

Steel Tank. The tank is built on grade in Contra Costa County approximately 20 km north of the City of Oakland. Most likely, this site experienced ground motions in the range of 0.05g to 0.10g. The steel roof beams were severely twisted. The damage is ascribed to wave sloshing forces from the Loma Prieta earthquake. For the vintage of this tank, it is unclear if the roof system had been designed to withstand any hydrodynamic forces (the main tank and its anchorage system were designed for earthquake forces).

A.1.6 Other Facility Damage

Damage to water treatment plants in the Loma Prieta earthquake included the following: loss of raw water pipelines; structural damage to buildings; damage to equipment (baffles, scrapers) in circular clarifiers and rectangular flocculation and sedimentation basis; loss of offsite electric power; damage to chemical injection tanks and equipment; and, loss of electrical equipment. The following paragraphs summarize the damage at specific locations.

Three of four clarifier tanks suffered damage at the Rinconada plant. (The fourth clarifier was empty at the time of the earthquake). The damage was ascribed to hydrodynamic impulsive and sloshing loads causing lateral overload of the central steel tower. Resultant damage included failure of radial launders, partial toppling of the central tower, jamming of scrapers. (PGA likely over 0.30g).

Similar circular clarifiers at a wastewater plant (San Leandro) did not suffer significant damage. (PGAs of about 0.15g). However, a chain-drive scraper system in a rectangular tank was damaged at the same site.

Scraper damage at a clarifier was reported at another water treatment plant 45 miles from the epicenter (PGAs about 0.05g or less).

In Oakland, six water treatment plants, varying in capacity from 35 MGD to 175 MGD, had no damage. All these plants experienced ground motions in the range of 0.05 to 0.10g. All had been designed with varying levels of seismic capacity, generally to the UBC code levels of the 1950s and 1960s. However, the raw water pipeline to one plant was damaged, and the plant had to be taken off line. At another plant, one of two parallel raw water lines leaked and had to be taken out of service; the plant was able to stay in service.

A.1.7 Service Impacts to Customers

The Loma Prieta earthquake demonstrated that the most significant cause of service loss to customers or to fire hydrants was due to buried pipeline damage. In small water systems with only tens of miles of pipe, only one or two pipeline breaks were enough to empty all storage tanks and de-pressurize the system, to make fire fighting impossible (San Francisco AWSS water system). Within the time frame needed for fire fighting purposes (first hours), manual operator actions were not fast enough to isolate the broken pipelines to allow the rest of the system to perform adequately. In large water systems with hundreds to thousands of miles of pipe, break rates under 0.03 per mile resulted in service disruption to under 3% of customers.

In the longer time frame after the earthquake, restoring service to customers for domestic water purposes was a time consuming activity. The critical activity needed to restore water service was the repair of underground pipelines. This activity is primarily a labor intensive effort, and water utilities often have limited manpower resources available to cope with many (hundreds) of pipeline repairs at the same time.

A.1.8 References

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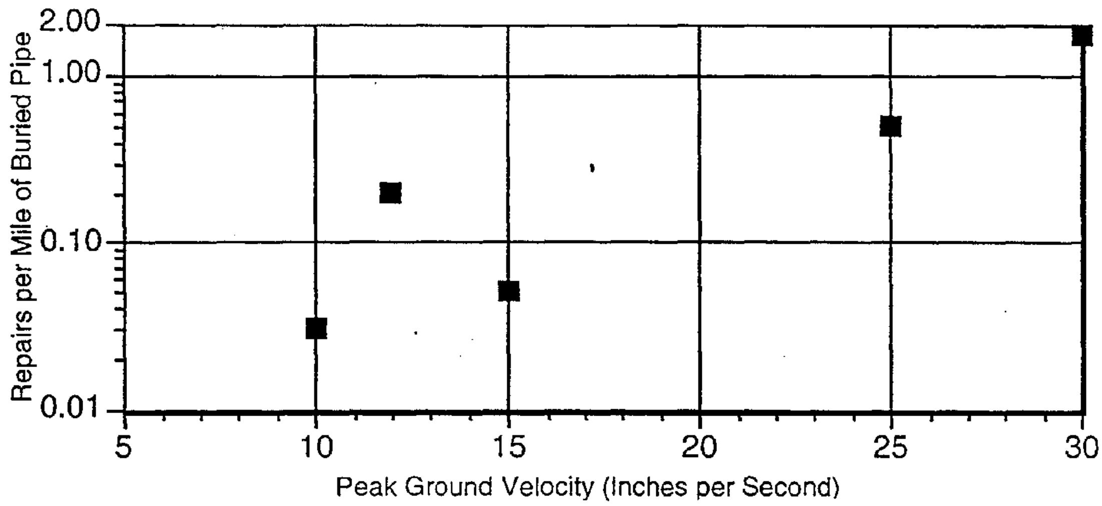

FIGURE A-1.1 – Pipeline Repair Rate Versus Generalized Peak Ground Velocity

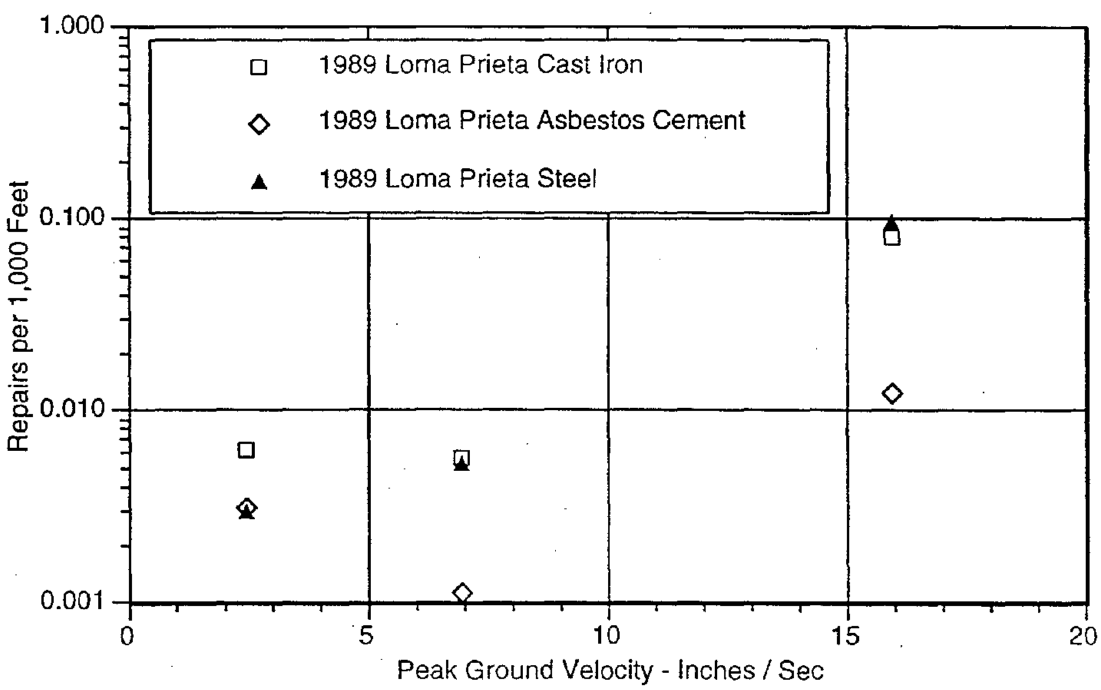

FIGURE A-1.2 – Pipeline Repair Rates by Material Type

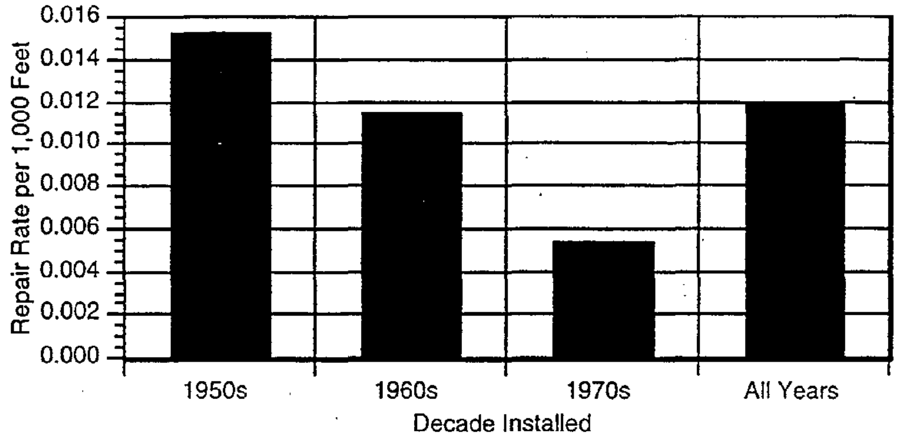

FIGURE A-1.3 – Steel Pipeline Repair Rates by Age

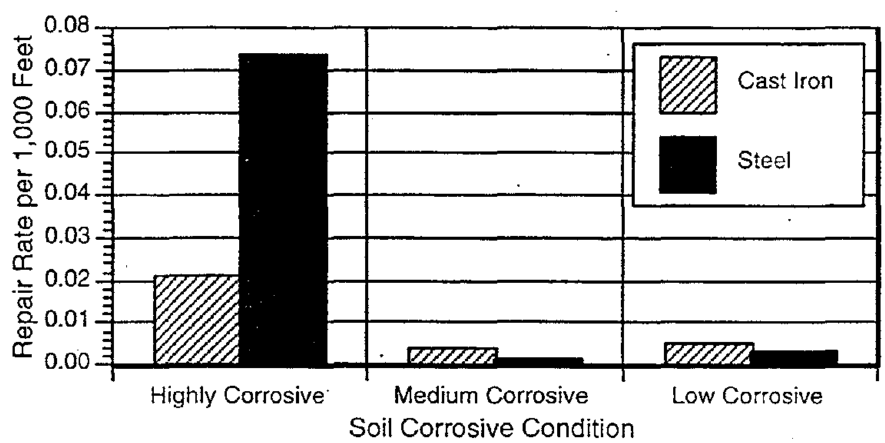

FIGURE A-1.4 – Pipeline Repair Rates for Cast Iron and Steel Pipe According to Relative Soil Corrosiveness

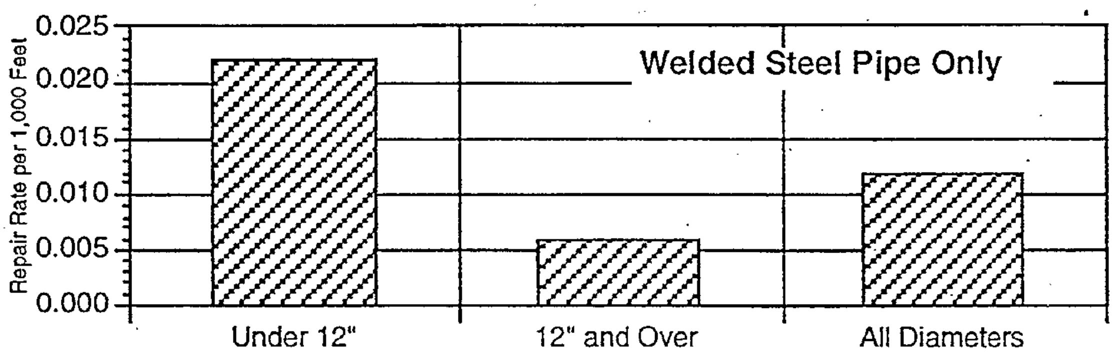

FIGURE A-1.5 – Pipe Repair Rates – Steel Pipe, by Diameter

A.1b 1989 LOMA PRIETA, CALIFORNIA EARTHQUAKE: WATER SUPPLY EFFECTS

By C. Scawthorn, S.E.

A.1.1 Introduction

This paper summarizes effects of the 1989 Loma Prieta earthquake on the components of the water supply systems, with emphasis on San Francisco. The earthquake and damage are first briefly summarized, followed by a discussion of the effects on water supply components, and an analysis of the interaction of water supply damage to the occurrence of fires.

A.1.2 Seismological and Overall Damage Aspects

On October 17, 1989 at 5:04pm local time, a M_s 7.1 earthquake occurred due to approximately 40 km rupture along the San Andreas fault. The epicenter of the 20 second earthquake was located near Loma Prieta in the Santa Cruz mountains about 16 km. NE of Santa Cruz, 30 km south of San Jose and about 100 km south of San Francisco (Figure A-1.1).

Major damage included the collapse of the elevated Cypress Street section of Interstate 880 in Oakland, the collapse of a section of the San Francisco-Oakland Bay Bridge, multiple building collapses in San Francisco's Marina district, and the collapse of several structures in Santa Cruz and other areas in the epicentral region. Damage and business interruption losses were estimated as high as $6 billion. Human losses were 62 people dead, 3,700 people reported injured, and over 12,000 displaced. At least 18,000 homes were damaged, 960 were destroyed and over 2,500 other buildings were damaged and 145 destroyed.

A.1.3 San Francisco Water Systems

San Francisco possesses three water supply systems:

- the Municipal Water Supply System (MWSS), owned and operated by the San Francisco Water Department (SFWD) and serving both fire fighting and municipal (potable water) uses
- the two others are specifically dedicated to firefighting use and are owned and operated by the San Francisco Fire Department (SFFD). These are the
    - Auxiliary Water Supply System (AWSS), first developed following the 1906 earthquake and fire and extended periodically thereafter, see Figure A-1.2, and the
    - Portable Water Supply System (PWSS), developed in the 1980's and primarily a truck-borne large diameter hose system.

The greatest damage to the water system consisted of approximately 150 main breaks and service line leaks. Of the 102 main breaks, over 90 percent were in the Marina, Islais Creek and South of Market infirm areas. The significant loss of service occurred in the Marina area, where 67 main breaks and numerous service line leaks caused loss of pressure, see Figure A-1.3.

A.1.4 Pipe Types and Performance

The MWSS and AWSS systems incorporate a variety of pipe materials and construction. The pipe can be roughly classified into 7 types: bell and spigot cast iron, double-spigot cast iron with heavy walls, ductile iron with rigid joints, ductile iron with flexible joints, riveted steel, gas welded steel, and arc welded steel.

- *Bell and Spigot Cast Iron Pipe:* The majority of MWSS water mains are of this type. Until the 1930's, pipe was vertical pit cast iron, 12 feet long, typically exhibiting brittle mechanical properties with tensile strengths on the order of 15 ksi (Ahmed, 1990). Joints were sealed with an oakum gasket and packed with lead caulk. Restraint was typically provided at dead ends and branch installations. After 1930, centrifugally cast iron came into wide use in water systems. Also brittle, this material nonetheless exhibits strengths on the order of 35 ksi. The advantage of stronger material may be offset to some degree by the stiffer joint construction to which SFWD switched at around the same time. Around the 1930s, SFWD began to seal joints with a rubber gasket and dry mortar caulk. Virtually all the water mains affected in the Marina District by the October 17, 1989 Loma Prieta earthquake were of the pre-1930 construction, as were all mains 24 inches in diameter or smaller in the 1906 San Francisco earthquake.

- *Ductile Iron Pipe with Rigid Joints:* After about 1960, ductile iron was used for MWSS pipe of 16 inch or smaller diameter, with geometry and construction similar to the cast iron it replaced, but cast centrifugally in 18 foot lengths. SFWD continued to use gasket and grout joints in new ductile iron pipe construction until 1989.

- *Ductile Iron Pipe with Flexible Joints:* Since 1989, SFWD pipe joints have employed U.S. Pipe's Field Lok Gasket, an elastomeric gasket fitted with mechanical teeth that provide longitudinal. The gasket requires no packing, thereby allowing significant rotational flexibility.

- *Riveted and Welded Steel:* Pre-1930 MWSS pipe larger than 24 inch diameter is of riveted iron or steel construction. Longitudinal joints were shop riveted; circumferential joints were riveted in the field. Gas welding came into use around 1930; during the 1940s, arc welding found widespread use. After about 1960, pipe larger than 20 inch diameter was of welded steel. Joints were bell and spigot with fillet welds at the lap joint. Pipe less than 24 or 30 inches in diameter received a single fillet weld on the outside. Pipe 30 or 36 inches in diameter or larger were welded inside and outside. Weld leg size was equal to pipe thickness.

- *1989 Pipe Damage and Ground Movements:* Breaks within the Marina were concentrated in the regions of hydraulic fill created circa 1906-1917. Locations of water main and service breaks are shown in Figure A-1.3, which is based on SFWD records.

A.1.5 Auxiliary Water Supply System Description And Performance

The AWSS consists of several major components:

- *Static Supplies*: The main source of water under ordinary conditions is a 10 million gallon reservoir centrally located on Twin Peaks, the highest point within San Francisco (approximately 750 ft. elevation).

- *Pump Stations*: Because the Twin peaks supply may not be adequate under emergency conditions, two pump stations exist to supply water from San Francisco Bay - each has 10,000 gpm at 300 psi capacity.

Both pumps were originally steam powered but were convertcd to diesel power in the I970's.

- *Pipe Network*: The AWSS supplies water to dedicated street hydrants by a special pipe network with a total length of approximately 120 miles. The pipe is bell and spigot, originally extra heavy cast iron (eg., 1" wall thickness for 12" diameter), and extensions arc now Schedule 56 ductile iron (eg., .625" wall thickness for 12" diameter). Restraining rods connect pipe lengths across joints at all turns, tee joints, hills and other points of likely stress.

- *Fireboat Phoenix*: The pipe network has manifold connections located at several points along the City's waterfront in order to permit the City fireboat Phoenix to act as an additional "pump station", drafting from San Francisco Bay and supplying the AWSS. The Phoenix's pump capacity is 9,600 gpm at 150 psi, about the same as Pump Station No. 2.

- *Cisterns*: Lastly, in addition to the above components, San Francisco has I51 - underground cisterns, again largely in the northeast quadrant of the City. These cisterns arc typically of concrete, 75,000 gallons capacity (about one hours supply for a typical fire department pumper).

The AWSS is a system remarkably well designed to furnish large amounts of water for firefighting purposes under normal conditions and contains many special features to increase reliability in the event of an earthquake.

The Loma Prieta earthquake resulted in only moderate shaking for most of San Francisco, typically of MMI VI, although selected areas sustained much greater shaking, perhaps as much as MMI IX in the Marina district. In the Marina district, 69 breaks in the domestic water supply and more than 50 service connections to water mains quickly dissipated all domestic water supply in the 40 blocks of the Marina district. The AWSS main serving the Marina district remained intact. However, as a result of the shaking, in locations other than the Marina, the AWSS sustained significant damage:

- The most significant damage occurred on Seventh between Howard and Mission Streets, where a 12 inch main broke. This location is on the boundary of Infirm Area No. 3 and moreover, the AWSS pipe at this location crosses over a sewer line. Soil settlements in this area arE thought to have occurred prior to and also as a result of the earthquake, causing the AWSS pipe to onto on the sewer line, and breaking.

- Other breaks included: (a) a break in an 8 inch hydrant branch, on Sixth between Folsom and Howard Streets (where the hydrant branch crossed up and over a sewer line) and (b) five 8 inch elbow breaks, four within Infirm Area No. 3 including one on Bluxome Street where a portion of a building collapsed onto an AWSS hydrant.

Major leakage resulted from these breaks such that Jones Street tank (controlling pressure for the Lower Zone) had completely drained in approximately 15 minutes. Leakage continued so that first arriving engines at the Marina fire found only residual water when they connected to AWSS hydrants, as described above. Due to uncertainty as to the number and location of AWSS breaks, valves connecting the Upper Zone to the Lower Zone were not opened, and Pump Stations I and 2, although available, were not placed in-service immediately but only at 8 PM, following identification and isolation of broken mains. As a result all pressure in the AWSS Lower Zone was lost for several hours following the earthquake. The pump stations were operated at half capacity so as to fill the AWSS mains slowly out of concern for entrapped air which was exhausted out of the Lower Zone through Jones Street tank (air could be heard exhausting from the tank). This operation continued until IO PM when full pressure was restored and Jones Street tank had been filled with salt water

Other damage was confined to:

- One 75,000-gallon cistern at Fifth and Harrison streets developed a leak at the cold joint between the roof and sidewall due to earthquake damage and lost 20 percent of its water, leaving 60,000 gallons for fire suppression purposes.

- Falling structures destroyed one High Pressure hydrant and damaged another.

A.1.6 Water System Performance and Firefighting

Twenty six fires occurred in San Francisco as a result of the earthquake. One of these fires occurred in the Marina District, and threatened to become a major conflagration. Firefighting efforts were severely hampered due to lack of MWSS and AWSS service to hydrants, due to the severe liquefaction and resulting pipe breakage in the Marina and elsewhere. Firefighters were forced to resort to drafting from nearby lagoons which however was inadequate, and the fire continued to grow. Deployment of San Francisco's PWSS in conjunction with the fireboat Pheonix provided the only adequate source of firefighting water, which was the only way the Marina fire was extinguished. It is worth noting the following:

- the Marina fire was potentially very severe - it was a very large fire in a dense neighborhood of wood frame construction - an unusually calm wind was a very fortuitous circumstance

- the fire was within 500 ft. of San Francisco Bay and the Pacific Ocean - the largest body of water on earth. However, these could not be drafted from by arriving fire engines, and the water was inaccessible.

- the MWSS system had over 400 million gallons of storage within San Francisco, but the numerous breaks in the Marina prevented adequate pressure or volume at Marina hydrants - elsewhere in the City, MWSS performance was generally satisfactory.

- the AWSS is designed for earthquake ground motions, and did not sustain damage in the Marina despite widespread liquefaction - nevertheless, it lost pressure in the Lower Zone due to breaks several miles away.

- the "backup to the backup" - that is, the PWSS backing up the AWSS which backs up the MWSS, provided firefighting water for extinguishment, at the Marina fire. The PWSS' flexibility and portability proved adequate to the task.

A.1.7 Summary

The 1989 Loma Prieta earthquake occurred 100 km from, and resulted in only moderate ground motions in, San Francisco. Nevertheless, significant damage occurred in areas of poor soils, particularly to buried piping, and only multiple redundancies in a special water supply system was capable of providing adequate firefighting water.

A.1.8 References

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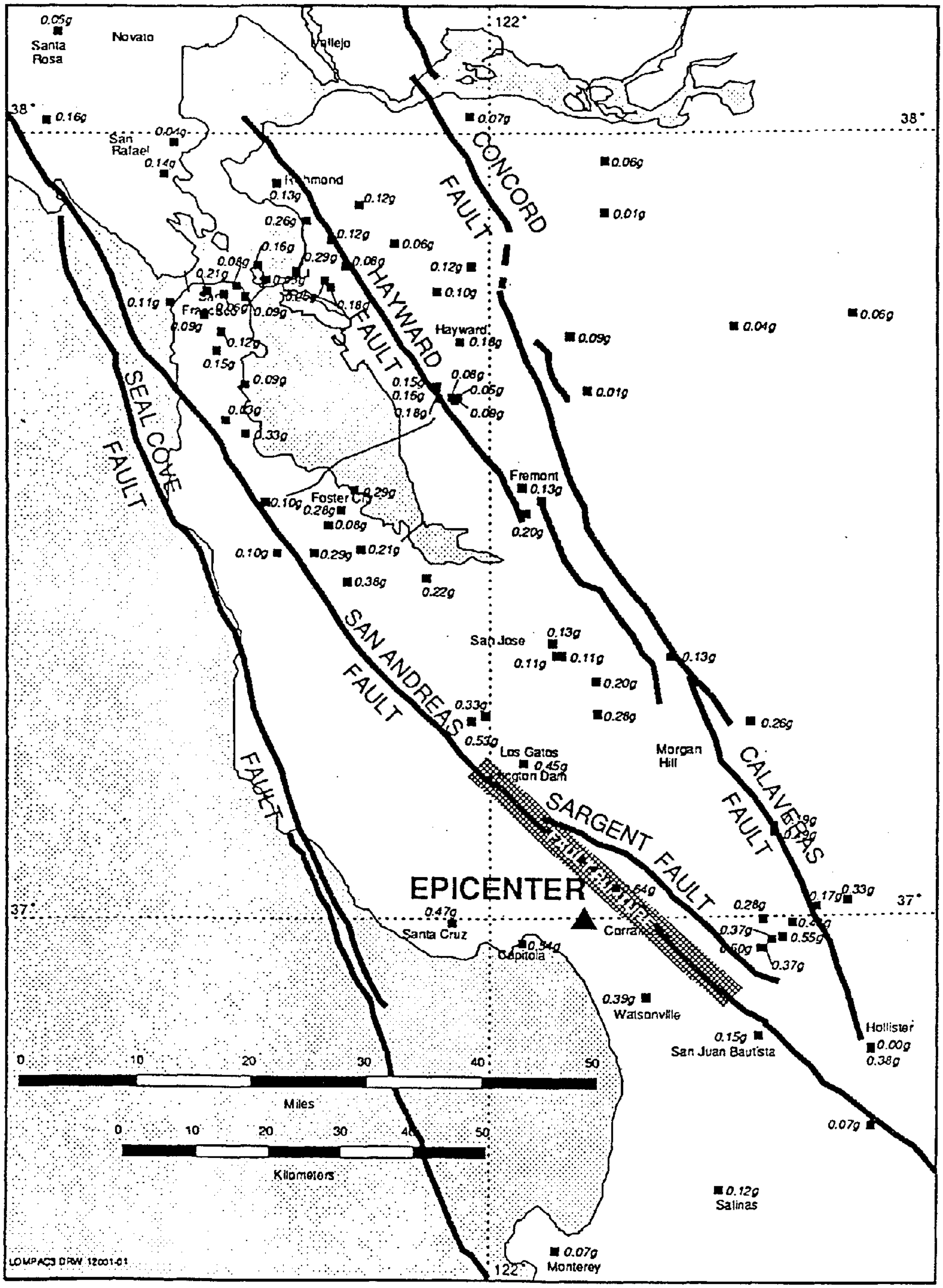

Figure A.1-1
Epicenter of the Loma Prieta Earthquake

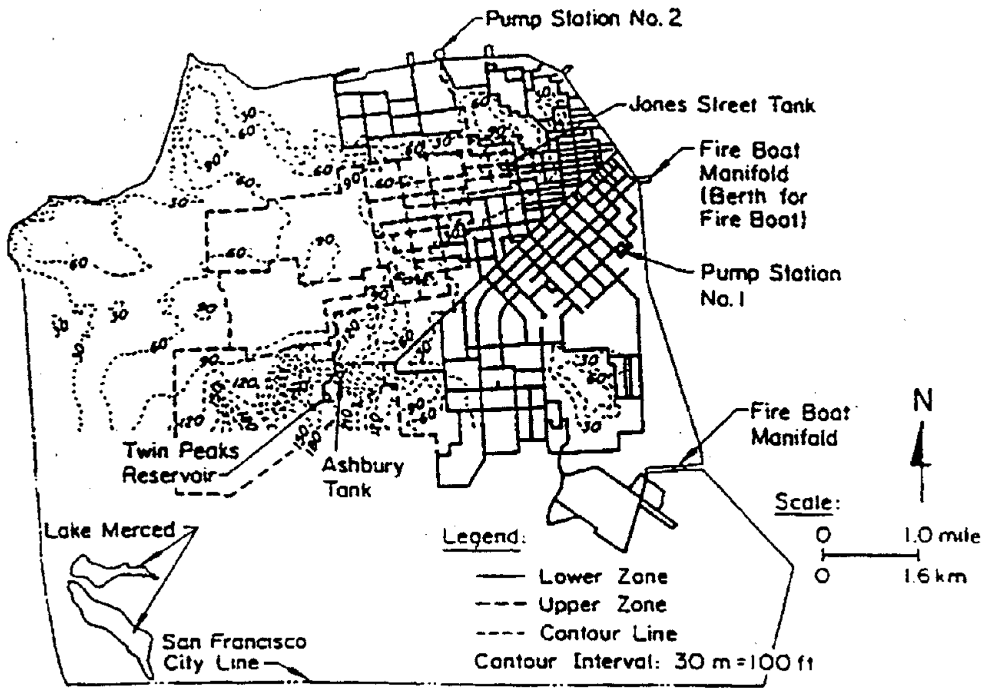

Figure A.1-2
Sketch Map of San Francisco Showing Location of AWSS Pipe Network
(After Scawthorn et al., 1990)

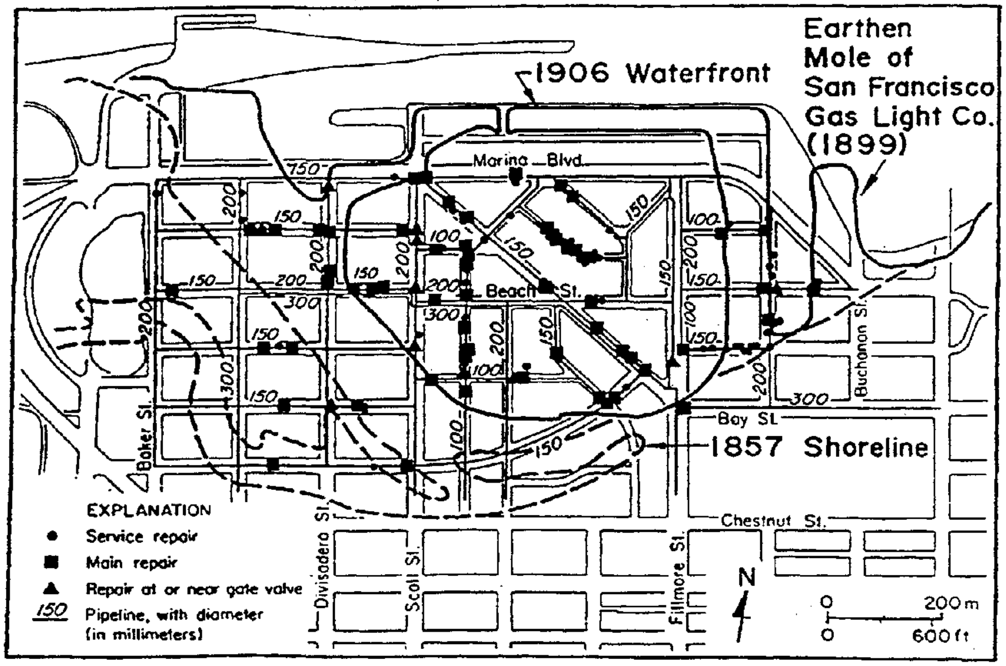

Figure A.1-3
Marina District Showing Locations of Repairs to MWSS Mains, Service Lines, and Sections at or near Gate Valves as of November 1, 1989
(Source: O'Rourke et al., 1992)

A.2 1994 NORTHRIDGE, CALIFORNIA EARTHQUAKE

By W. Heubach

The areas where water supply was most affected by the January 17, 1994 Northridge Earthquake, a magnitude 6.7 event, were the areas nearest the epicenter: the San Fernando Valley, Simi Valley and the Santa Clarita Valley (see Figure A-2.1). Water for these areas is provided by aqueducts that deliver water from the Sacramento/San Joaquin Delta, the Owens River and the Colorado River, and local groundwater sources. At the time of the earthquake (approximately 4:31 AM, Pacific Standard Time), the water treatment facilities for these regions were the Metropolitan Water District's (MWD) Joseph Jensen Water Treatment Plant, the Los Angeles Department of Water and Power's (DWP) Los Angeles Aqueduct Water Filtration Plant and the Castaic Lake Water Agency Water Treatment Plant.

A.2.1 Component Performance

Extensive damage occurred to many water system components located in the San Fernando, Simi and Santa Clarita valleys. In addition to intense ground shaking where some locations experienced peak ground accelerations in excess of 1.0g, permanent ground displacements from ground lurching, rupture and liquefaction were also observed in several areas.

A.2.1.1 Sources

The San Joaquin/Sacramento Delta, Owens River and Colorado River are all located 150 miles or more from the epicentral region. Consequently, these surface water sources were not affected by the earthquake. Local groundwater well damage was reported to have been minimal. Because commercial power was lost for periods ranging from approximately 12 hours to three days (when power was restored 99.5% of the customers), source disruption occurred at well facilities without back-up power.

A.2.1.2 Transmission Lines

Six transmission pipelines used to transport surface water to or from the treatment plants sustained damage during the Northridge Earthquake. The Castaic Conduit, a modified prestressed concrete cylinder pipe, transmits water from the Castaic Lake Water Agency Water Treatment Plant at Castaic Lake (the terminus of the West Branch of the State Water Project) to the Santa Clarita Valley. Due to damage that resulted in 35 leaks, the Castaic Conduit was out of service for 67 days after the earthquake. During the repair period, local groundwater sources were used to supply the Santa Clarita Valley.

The Foothill Feeder, a welded steel line that transmits water from Castaic Lake to the Jensen Water Treatment Plant, was also damaged where the 170-inch diameter line splits into two 85-inch treatment plant inlet lines. A circumferential crack on a bell of the eastern 85-inch line was repaired in two days after the earthquake. The MWD East Valley Feeder and pumping station were activated so that water could be provided to the affected areas in the San Fernando Valley from the Colorado River Aqueduct/State Water Project - East Branch.

DWP's Los Angeles Aqueduct No. 1 and No. 2, which supply water for the DWP's Los Angeles Water Filtration Plant from the Owens River, were also damaged. Aqueduct No. 1 was damaged at welded steel and reinforced concrete siphons. Damage to Aqueduct No. 2, a welded steel pipeline, shut down the line for 12 days after the earthquake. During the shutdown of Aqueduct No. 2, Aqueduct No. 1 was operated at a low flow rate. After temporary repairs were made to Aqueduct No. 2, Aqueduct No. 1 was shutdown for two months while temporary repairs were made. Additionally, both aqueducts were shutdown in the last week of February and first two weeks of March while repairs were made to the open concrete channel inlet to the Los Angeles Water Filtration Plant. Permanent repairs to Aqueducts Nos. 1 and 2 were completed in the summer of 1994. In addition to reliance on the redundancy provided by the two aqueducts, reservoir storage, groundwater supplies and the Colorado River Aqueduct/State Water Project - East Branch were used as alternate supplies during the outages (Earthquake Engineering Research Institute, 1995).

The 78-inch North Branch Feeder and 51-inch Callegus Conduit, both prestressed concrete cylinder pipes that supply Simi Valley from the Jensen Treatment Plant, were also damaged. The 78-inch line received 15 to 20 major pulled joints and approximately 500 cracks. The 51-inch Callegus Conduit incurred damage at three blind flanges and to bolts at future service locations. Although the Callegus Conduit was able to remain in intermittent service, the North Branch Feeder was out of service for 6 weeks until repairs were completed. Reservoir storage and groundwater were used as alternate sources during the shutdowns.

A.2.1.3 Water Treatment Plants

There was a significant amount of nonstructural damage and some sloshing and settlement damage at the 550 MGD Jensen Water Treatment Plant, the 600 MGD Los Angeles Water Filtration Plant and the 25 MGD Castaic Lake Water Agency Water Treatment Plant. Most damage could be easily repaired and did not significantly affect plant performance. However, loss of power and transmission line damage upstream and downstream of the plants shut the plants down for 36 hours (EQE, 1994).

A.2.1.4 Reservoir Performance

Approximately three dozen water storage reservoirs lost functionality during the earthquake. Failure of inlet/outlet piping caused by tank uplift was the primary reason for loss of functionality. Most damage occurred in steel tanks that did not meet AWWA seismic provisions. Because the earthquake occurred early in the morning, tanks were much more likely to have been full than if the earthquake had occurred in the late afternoon or evening. Tank failure effects on water system operation were not documented.

A.2.1.5 Pumping Station Performance

Structural and nonstructural damage to pump stations was not reported. However, some stations did not have emergency power and were not functional due to loss of commercial power. There is speculation that water pressure restoration at some fire hydrants that initially lost water pressure occurred when pump stations that lost commercial power were switched over to emergency power sources.

A.2.1.6 Distribution Pipelines

Distribution pipe damage was the major contributor to water unavailability following the Northridge Earthquake. Over 1400 repairs were made in the San Fernando Valley and over 300 repairs were made in the Santa Clarita and Simi valleys (Lund and Cooper, 1995). Liquefaction/lateral spread, ground lurching and ground rupture were apparently responsible for most of this damage. In addition to pipe damage, air release and vacuum valves were sheared from ground shaking and some fire hydrants ruptured (possibly from water hammer).

Because the distribution pipe damage was so extensive, large areas lost water pressure. In addition to widespread loss of water pressure, the threat of contamination from infiltration resulted in boil water orders that were not lifted until January 24 in Simi Valley, January 29 in Los Angeles and February 4 in the Santa Clarita Valley.

A.2.2 Component Failure Impact on Water Availability

The primary reason for loss of water pressure was the extensive damage suffered by distribution system piping. In addition to loss of water pressure, the threat of infiltration through damaged lines resulted in boil water orders that lasted over two weeks in some areas.

Damage to transmission piping, reservoirs and pump stations had less of an impact on system performance than the broken distribution lines. Because there were multiple sources of water supply and multiple transmission lines, those transmission lines that remained functional and local groundwater sources were able to provide a large portion of the water supply that was made unavailable by transmission line failure.

The impact that reservoir failure had on system operation was not documented. Loss of functionality at pump stations resulted in loss of pressure in some zones dependent upon pumping. In some instances, fire department pumpers were used to pump water from hydrants to zones reliant on nonfunctional pumping stations.

A.2.3 Performance of Auxiliary Systems

The regions affected by the Northridge Earthquake did not have permanent auxiliary water distribution systems. In areas where water pressure was lost, drinking water was provided by tank trucks. Emergency fire fighting water was provided by a variety of sources that included fire department tankers, nearby swimming pools and other sources that could be used for drafting, and even United States Forest Service water-dropping helicopters.

A.2.4 Post Earthquake Water Availability

Figure A-2.1 shows the extent of water outages following the Northridge Earthquake. Although permanent repair of damaged facilities continued several months after the earthquake, most water services were restored within 7 days of the earthquake (EQE International and the Office of Emergency Services of the State of California, 1997, to be published).

A.2.4.1 Fire Suppression Water Availability

Several significant fires were caused by gas lines rupturing (Todd, 1994). Because water system components and natural gas pipelines and services are both most likely to be damaged in areas of permanent ground displacement and/or severe ground shaking intensity, fire locations often correspond to areas that are without water pressure.

The Los Angeles Fire Department reported a lack of water pressure at hydrants in much of the western and northern areas of the San Fernando Valley. The Balboa Boulevard fire, which received significant coverage by the media, was controlled by drafting water from nearby swimming pools. It took 70 minutes and 14,000 gallons of water to control this fire.

Other fires were also controlled by using swimming pool water. Fire department tank trucks were used as supplemental sources. United State Forest Service helicopter water drops were also used on building fires.

In some cases, hydrants that were initially without pressure regained pressure within an hour or two after the earthquake. In other cases, hydrants lost pressure during the fire suppression effort. At one trailer park where water was not available, the responding companies left to answer other calls after they verified everyone had been evacuated. When the companies returned an hour later, hydrant pressure had recovered to 30 psi. The reason for temporary loss of pressure is not known for sure but it is speculated that water pump stations that lost power may have regained either commercial power or were switched over to emergency power sources.

A.2.4.2 Potable Water Availability

In areas where distribution pipes were damaged, water was supplied by tanker trucks. These trucks were provided by local water, beer and softdrink bottlers, the U.S. Army, the California National Guard, and local contractors. At the peak of the effort, tankers were supplying over 100,000 gallons of water per day.

In some places such as the Santa Clarita Valley, groundwater was used. Because transmission lines to/from treatment plants were damaged, the groundwater was chlorinated. Boil water orders, which lasted for up to 12 days in some places (Dames & Moore, 1994), were also in effect as a precaution against possible contamination from infiltration.

A.2.5 References

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Figure A-2.1
Areas Without Water Service After the Northridge Earthquake

A.3 1995 GREAT HANSHIN (KOBE), JAPAN EARTHQUAKE

By D. Ballantyne

A.3.1 Earthquake Overview

The Great Hanshin (Kobe) Earthquake occurred at 5:46 AM on January 17, 1995. The magnitude 6.9 event, as measured by the USGS, heavily impacted an area approximately 40 km along the causative fault, with the most severe damage occurring within 3 km on either side of the fault. Peak ground accelerations were in the order of 0.8g.

The death toll exceeded 6,000 people, over 500 of whom were killed by fire while trapped in their collapsed houses, and 300,000 people were left homeless. Loss estimates exceeded $100 billion.

A.3.2 Overview of Water System

A.3.2.1 System General Overview

The City of Kobe system serves 610,000 households including 1.5 million people. The system was started in 1900. The balance of the City is mountainous and undeveloped. The Kobe Water system includes:

- 4,000 km pipe
- 240 reservoirs at 119 sites
- 43 pump stations with 208 pumps
- 57 km transmission tunnels

Kobe's water is pumped from surface water supplies with primary sources including Lake Biwa and the Yodogawa River. It is pumped to the major water purification plants. Treated water runs by gravity to the west through a tunnel system at an elevation of 90 m, and feeds the urban areas below. Water used at an elevation above 90 meters is pumped from the tunnel system.

All Kobe's water is treated before it is distributed. There are eight water purification plants treating potable water for Kobe, and one treating water for industrial use. The three largest plants are Uegahara, Hanshin, and Sengari.

The tunnel system operates in the open channel flow or low pressure regime. The first tunnel, 1.8-meter diameter, was constructed in the 1930's in rock with an unreinforced concrete lining. The second rock tunnel, 2.5-meter diameter and built in the 1950's, is lined with reinforced concrete.

The Kobe potable water system is designed primarily for drinking, but also provides water for fire protection. There is no dedicated water system for fire suppression. There are approximately 900, 40 cubic meter cisterns located throughout the City for use for fire suppression.

A.3.2.2 Distribution Pipe

Kobe has had an aggressive pipeline replacement program that has resulted in nearly 90 percent of their system being constructed of steel or ductile iron pipe. Between 1978 and 1991 they replaced approximately 300 km of pipe.

Approximately 6 percent of their ductile iron pipe employs special seismic joints. These joints, Japan Water Works Association, JWWA, Standard G 113 and 114, are specially designed to allow longitudinal extension and compression but are restrained before they separate. The ductile iron pipe with S joints is manufactured by both Kubota and Kurimoto, and is estimated to cost 30 percent more than pipe without seismic joints. All new pipelines greater than 400 mm in diameter and all pipelines in reclaimed and landslide areas are required to be constructed with this seismic resistant pipe.

The steel pipe Kobe uses employs butt welded joints. Pipe larger than 600 mm diameter is welded from both the interior and exterior; smaller diameter pipe only from the exterior.

A.3.2.3 Distribution Reservoirs and Pump Stations

The Kobe Water Department has an inventory of 240 service reservoirs at 119 sites. These include cast-in-place concrete (approximately 80 percent), and precast concrete and welded steel (approximately 20 percent combined), and range in capacity from 40 to 20,000 m^3.

Nearly all the reservoirs are constructed with two hydraulically independent tanks at each site. Many of these are single tank structures constructed with a dividing wall. This provides redundancy for post-earthquake function and allows removing one from service for cleaning without impacting service. Kobe does not use pressure reducing valves between pressure zones as are commonly used in the United States. Rather, they use control tanks, with flow from the upper pressure zone into the tank controlled by an altitude valve.

A.3.2.4 Earthquake Monitoring and Control

Kobe Water has an earthquake monitoring and control system to isolate water in selected reservoirs from the system to maintain it for drinking following an earthquake. The system is not designed to save water for fire suppression.

The system consists of earthquake ground motion monitoring at the Okuhirano Control Center, a control panel, telemetry, and reservoirs with earthquake isolation valves at 21 locations. The earthquake valves are motor operated with backup batteries.

The earthquake control system allows both automated and manual control of isolation valves on service reservoirs. The decision logic is as follows:

<u>Peak Ground Acceleration Control Thresholds</u>

40 gal - manual alarm (980 gal = 1g)
80 gal - automatic shut down when combined with excessive rate of change of flow
250 gal - automatic shut down

There are dual reservoirs at each of 21 sites; one has an isolation valve to be controlled following an earthquake, and one does not. This concept allows shutdown of one reservoir while maintaining service should the second reservoir inadvertently shutdown. If the system can keep up with system leakage, the isolated reservoir can be put back on line from the Control Center. If the system can not keep up with the demand, reservoir remains isolated.

Emergency planners based the number of reservoirs with isolation valves on the capacity required to provide the population 3 liters/day for drinking for 7 days. Planners estimated that it would take up to 7 days to repair the system. Isolated tanks were selected so that each would serve an area approximately 4 km in diameter.

A.3.3 Conditions Following the Earthquake

Earthquake valves closed on 18 of 21 service reservoirs following earthquake storing 33,800 m^3 of water.

Two thirds of the 86 distribution reservoirs serving the urban area without earthquake valves emptied in the first 6 hours, and the balance within the first 24 hours. This resulted in inadequate water available for fire suppression of the 350 fires that broke out in the first two days following the earthquake.

Water carried to the Hanshin Purification Plant was reduced from 6,000 to 1,000 m^3/hr immediately following the earthquake. This was caused by loss of power to Yodogawa River Pump Stations for 15 hours following the earthquake as well as transmission main breaks.

Emergency supply using water tank trucks was initiated on the evening of the day the earthquake occurred.

The following day, water restoration began. Leaks were identified and repaired with the following priorities:

1. Damaged areas, medical facilities, areas with large populations.
2. Broken pipes, minor leakage areas (that is fix areas that first that could be easily restored).
3. Broken pipes, major leakage areas.
4. Shelters and areas in danger of landslides.

A.3.4 Damage to Water Facilities

A.3.4.1 Water Purification Plants

The Uegahara, Hanshin and Motoyama water purification plants were damaged. The Sengari, Okuhirano, and Sumiyoshi water purification plants had little damage.

Uegahara Water Purification Plant - The plant was rendered inoperable following the earthquake. On the west side of the plant, fill behind a retaining wall subsided approximately 30 cm when the retaining wall moved outward resulting in separating pipeline joints on three treated water lines in an area 100 m long.

The solids handling building was located in the same area that subsided. The floor settled differentially. There was subsidence around pile supported foundation of sludge hoppers. Unanchored electrical equipment toppled in sludge thickener building.

This same subsidence also probably led to the following damage:

- Clarifiers - damage to expansion joints; water leakage
- Process Tanks - some leakage

- Sludge Thickening Equipment - extensive damage to piping and equipment
- High Rate Filter Piping - bent and disconnected
- Other - damage to pipe, mechanical, and electrical equipment.

Steel baffles in the clarifiers were damaged from impulsive hydraulic forces.

Hanshin Water Authority Water Purification Plant - The plant was not damaged. There was a major landslide, the largest slide in the urban area in the Kobe Earthquake, in back of the treatment plant, undercutting the wastewater treatment building which was pile supported, and destroying a tennis court and a sludge drying bed. The slide killed 36 people in a housing development below, and dammed the Nigawa River.

Motoyama Water Purification Plant - The Motoyama Plant had damage to two segments of raw water pipelines and minor cracking to a process tank. Pipelines to and from the backwash water tank failed as a result of slope failure.

A.3.4.2 Intake, Pump Stations, and Transmission Pipelines

The intakes in and pump stations near the Yodogawa River were not damaged. The two major lines from the Yodogawa River were broken in 10 locations. One 100 meter section had to be replaced. It took 12 days to repair the raw water lines serving the plant.

The Mt. Rokko Tunnels, carrying water from the Uegahara and Hanshin plants west, to distribution, had spalled concrete, but remained functional.

Pump stations were undamaged. Damage to air release valves was reported.

A.3.4.3 Distribution Reservoirs (Tanks)

Of the 119 reservoir sites throughout the City, there was damage to one pre-World-War-II cast-in-place partially buried concrete tank, where there was a crack on the bottom and side.

At the Egeyama Reservoir, there was leakage in the pipe connecting the reservoir and the adjoining well, and a vertical crack in the expansion joint. A 500 mm diameter pipe 10 meters long was replaced.

A.3.4.4 Distribution Pipes

The pipeline systems damaged in the earthquake, reported on in this section serve Kobe, Nishinomiya, and Ashiya (Shirozu et al, 1996). There were 2033 pipeline failures, and 572 appurtenance failures. There was significant exposure and related damage to ductile iron, cast iron, PVC, and asbestos cement pipe. This is the first earthquake with this level of pipe exposure where the repairs have been documented in great detail. Net failure rates of the various pipe materials were (repairs/km): ductile iron - 0.49, cast iron - 1.51, PVC - 1.43, welded steel - 0.47, and asbestos cement - 1.79. Refer to Table A-3.1 for a summary of pipeline failure modes for various pipe types.

Pipeline damage observations include:

- Of the 1257 joint pull out failures for all pipe, 677 of them, over 1/2, occurred in areas where there was no liquefaction. Refer to Table A-3.2 for a summary of pipe failure modes for three categories of liquefaction extent.

- Joint pull out was not pipe diameter dependent.

- Joint pull out was the most frequent pipeline failure mode (1257 out of 2033 failures) and is common to all non-restrained bell and spigot pipes. Non-pull out failure mode pipe failure rates, including pipe barrel bending and joint failures, were (repairs/km) DIP - 0.02, CIP - 0.70, PVC - 0.91, welded steel - 0.43, and AC - 1.38. These failure rates are indicative of material ruggedness. These repair rates exclude fitting failures as their material is unknown.

- Pipe bending, and to a lesser degree, fitting failure repair rates were smaller for larger diameter pipe.

- There were only 10 ductile iron pipe barrel and fitting failures in 1874 km of pipe (damage rate of 0.00). 96% of ductile iron pipe failures were joint pull out.

TABLE A-3 .1 - Summary of Pipeline Failures by Failure Mode and Pipe Type

	Failure Rates/km - Number of Failures													
Failure Mode	**DIP**		**CIP**		**PVC**		**Steel**		**AC**		**Other**		**Total**	
Pipe length (km)	1874		405		232		30		24		116		2681	
Barrel	0	9	0.63	257	0.38	88	0.33	10	1.24	30	0.15	17	0.15	411
Fitting	0	1	0.31	124	0.17	40	0.03	1	0.04	1	0.04	5	0.06	172
Pulled Joint	0.47	880	0.49	199	0.33	76	0	0	0.37	9	0.8	93	0.47	1257
Joint Failure	0	2	0.06	25	0.5	115	0.07	2	0.08	2	0.03	4	0.06	150
Joint Intrusion	0	5	0	1	0.01	3	0	0	0	0	0	0	0	9
Joint Unknown	0.01	14	0	1	0.02	4	0.03	1	0.04	1	0	0	0.01	21
Other	0	4	0.01	4	0.02	5	0	0	0	0	0	0	0	13
Total	0.49	915	1.51	611	1.43	331	0.47	14	1.79	43	1.03	119	0.76	2033

TABLE A-3 .2 - Summary of Pipeline Failures by Failure Mode and Liquefaction Extent

	Failure Rates/km – Number of Failures							
Failure Mode	**0% Liquefaction**		**50% Liquefaction**		**100% Liquefaction**		**Total**	
Pipe length (km)	2060		388		232		2681	
Barrel	0.13	261	0.25	98	0.22	52	0.15	411
Fitting	0.04	86	0.17	67	0.08	19	0.06	172
Pulled Joint	0.33	677	0.76	297	1.22	283	0.47	1257
Joint Failure	0.04	88	0.13	52	0.04	10	0.06	150
Joint Intrusion	0.00	4	0.01	3	0.01	2	0	9
Joint Unknown	0.01	14	0.02	6	0.00	1	0.01	21
Other	0.00	10	0.01	3	0.00	0	0	13
Total	0.55	1140	1.35	526	1.58	367	0.76	2033

The three predominant types of service line failures were: 1) house collapse, 2) joint separation, and 3) failure where the line passed below the concrete stormwater gutter. Areas of significant differential settlement between pile-supported and non-pile-supported pipe also resulted in damage to pipe connections to buildings. Service line repairs were being accomplished at a rate of 2 per day with a crew of 5 to 6 workers.

There was significant pipeline damage on Kobe's Port Island where Tyton-type and/or mechanical joints were deployed. In contrast, there was no water pipeline damage on Rokko Island where the great majority of mains were ductile iron pipe with S-type joints.

There were two major issues identified that had delayed system restoration:

1. No water pressure was available to check the repairs while the tunnels remained out of service.
2. Access - limited by collapsed buildings and traffic congestion.

Pipelines hung on the Kobe, Rokko, Mikage, and Fukae bridges were damaged, and in some cases disrupted service to these islands.

A.3.4.5 Water Department Buildings

The Water Department Main Office, including administration and engineering, was located on the 6th floor of the Old City Hall Building (City Hall Annex). The 6th floor of the 8 story concrete frame building pancaked. Because of the early morning hour, only one employee was inside, and was killed. Distribution system maps, facility drawings, etc. were not available during the initial restoration phase.

The Water Department maintained a distribution and maintenance buildings in five wards throughout the City. The upper three floors of the eight story Water Department Eastern Center partially collapsed. The Western Center partially burned. The unburned section was being used. The combined damage of these three facilities made emergency response and restoration very difficult. The Central, North, and Tarumi Centers were not damaged.

A.3.4.6. Monitoring and Control System

The control system designed to isolate 21 reservoirs for drinking worked at 18 locations, and water was saved for drinking. Two of the failures were electrical or mechanical and one was hydraulic. The three repeater stations and telemetry systems were undamaged.

A.3.4.7 Summary

The Great Hanshin Earthquake left one million households without water following the event, with only 80 percent restoration in one month. Water for fire suppression was substantially exhausted after six hours.

The two large diameter transmission lines carrying water from the Yodogawa River to the Uegahara and Hanshin water purification plants failed in ten locations. This source constitutes nearly three-quarters of Kobe's supply. The Uegahara and Hanshin plants are Kobe's largest. In addition to their supplies being disrupted, soil failures at the plants resulted in failure of the major treated water lines, as well as moderate damage at the plants themselves.

The tunnels carrying water from the plants west to Kobe remained functional but required repair in the months following the earthquakes. Kobe Water had an aggressive earthquake design program for at least some of their service reservoirs. There was one tank failure reported for their 240 tanks at 119 sites. There was one connecting piping failure at one tank.

Nearly 90 percent of Kobe's water distribution system was made up of either welded steel or ductile iron pipe, both thought to be resistant to earthquakes. This unusually high percentage of pipe resulted from an aggressive pipeline replacement program (non-earthquake related) initiated in the mid-1960's. Even with this in place, an approximately 2,000 pipeline failures occurred in distribution piping. Much of this damage is associated with joint separation associated with both liquefaction/lateral spreading and ground motion in the areas of the strongest ground shaking.

Kobe had implemented a monitoring and control system designed to isolate reservoirs following an earthquake to be used for drinking. The system worked as designed. It needs to determine if they would have been better off focusing their mitigation efforts on water for fire suppression.

There was no system focused on post-earthquake performance to provide water for fire suppression. There was no water available to put out many of the fires.

The lifeline earthquake community in the U.S. should learn from both the successes and failures observed in Kobe. Successes include the monitoring and control system, performance of water treatment plants (except geotechnical), tankage, and seismic joint pipe. Failures include lack of adequate water supply for fire suppression (including alternate supplies), and inadequate focus on geotechnical issues.

A.3.5 References

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A.4 1992 LANDERS AND BIG BEAR, CALIFORNIA EARTHQUAKES

Summary By M.J. O'Rourke

The Landers earthquake (magnitude 7.3) occurred at 4:58 AM on June 28 in a sparsely populated area of the Mojave desert. It was followed 3 hours later by the Big Bear earthquake (magnitude 6.5), which occurred near Big Bear Lake in the San Bernardino mountains.

A.4.1 System Description

The greater Yucca Valley/Joshua Tree area of the Mojave dessert consists of small towns and rural areas with low population density. It is serviced by three main water systems. The sources for all three systems are groundwater basins. Hence, the components are as follows: wells, tank storage, pumps and distribution piping. For example, the Hi Desert Water District, which services about 10,000 families, has 17 tanks with capacities ranging from 10,000 to 2,000,000 gallons, 17 wells and pumps and roughly 300 miles of buried piping (roughly 40% steel and 60% asbestos cement). On the other hand, the Big Horn Desert View Water Agency (approximately 1,600 service connections) has 5 wells, three tanks with total storage capacity of 0.7 MG and roughly 100 miles of buried piping (mostly asbestos cement) with diameters of 6 inches or larger.

A.4.2 System Performance Vis-a-Vis Fire Suppression

The most significant damage to water systems was leaks and breaks in water lines and damage to storage tanks. There were approximately 600 seismically induced pipe repairs, at least several hundred of which occurred in the fault rupture zone. At least two tanks suffered damage. Customers had low pressure or were without water for several hours to two weeks.

A.4.3 Component Performance

Available information on component performance and associated impacts on water supply/fire fighting capability are summarized.

- **Distribution Piping** - Reported damage rates for two of the water providers were: a "couple of hundred" repairs in roughly 100 miles of 6 inch and larger distribution piping, and 200 to 250 repairs in 300 miles of pipe.

- **Wells and Pumps** - There was no reported seismic damage to the several dozen wells and booster pumps in the area. However, commercial power was disrupted.

- **Storage Tanks** - At least two water storage tanks suffered significant damage and were subsequently replaced. The first is a 0.2 MG bolted steel tank (diameter, D ~ 38 ft; height, H ~ 24 ft) which experienced an elephant's foot buckle at its base and a severed inlet/outlet pipe. Temporary restoration was accomplished by "by-passing" the tank (i.e., direct connection between the on-site wells and the nearby booster pump). The second was a 0.4 MG welded steel tank, which suffered a tank wall tear due to elephant's foot buckling and a severed inlet/outlet pipe. Temporary repair by a mutual aid agency put the tank back in service at about half capacity.

A.4.4 References

The information presented above is based upon “Lifeline Performance in the Landers and Big Bear (California) Earthquakes of 28 June 1992,” by La Val Lund, *Bull. Seis. Soc. Am.*, v.84, n.3, p 562-572, as well as field notes from a post-earthquake reconnaissance visit by Michael O’Rourke.

A.5 1992 PETROLIA, CALIFORNIA EARTHQUAKES

Summary By M.J. O'Rourke

A.5.1 System Description

A series of three large earthquakes (magnitudes 6.9, 6.2, 6.5) on April 25 and 26 affected a region consisting of small town and rural areas with low population density. The sources of water in the affected area are groundwater and surface water. For the Eureka area, the Humboldt Bay Municipal Water District stores surface runoff in the Roth Lake Reservoir (16,000 MG), located behind the Matthews Dam (compacted earth fill built in 1962) on the Mad River. The reservoir feeds the Mad River, on which five pump wells are located. The river water is pumped through an "out-of-service" treatment plant, into a 20 MG storage reservoir and then out to the distribution system. In other areas, groundwater is pumped from wells into storage tanks and then to the distribution network, apparently without extensive treatment. There is a dual distribution system in Scotia, a high pressure system for fire fighting, and a lower pressure system for domestic supply.

A.5.2 System Performance Vis-a-Vis Fire Suppression

Seismic damage to water mains in the high pressure (fire fighting) system in Scotia resulted in loss of fire suppression capability. The Scotia fire equipment was not capable of drafting from the low pressure domestic supply system, and a fire destroyed a small strip mall.

The City of Rio Dell lost water supply due to damage of an 8 inch main which resulted in their three storage tanks draining. Temporary fire protection was provided by tank trucks, which fortunately were not needed due to an absence of ignitions.

A.5.3 Component Performance

Available information on component performance and associated impacts on water supply/fire fighting capability are summarized.

- **Distribution Piping** - One of the two pipe breaks in the Eureka water system was in a 6 inch lateral for a K-Mart. The lateral feeds one of the three fire hydrants at the site. A valve was used to isolate the break.

  The City of Rio Dell lost water supply due to a break in an 8 inch main. The break occurred in a riser at the abutment of a highway bridge which carried the main over the Eel River. The break caused the systems three water tanks (two 0.25 MG steel tanks, and one 25,000 gallon redwood tank) to drain.

  Similarly, distribution pipe damage in Ferndale caused partial drain down of a 1.0 MG tank.

- **Wells and Pumps** - Wells operated normally and there was no reported seismic damage to pumps. However, commercial power was lost at some pump stations, ranging from minutes to hours.

- **Storage Tanks** - There was no reported direct seismic damage to storage tanks. However, as noted above, some tanks drained due to distribution pipe damage.

- **Water Treatment Plants** - No damage was reported at the out-of-service treatment plant. However, the settlement tanks and clarifiers were empty.

- **Reservoir/Dam** - Inspection of the dam indicated no seismic damage.

A.5.5 References

The information presented above is based upon a December 1992 draft of "Response of Lifelines and Their Impact on the Emergency Response to the Petrolia Earthquakes", April 25-26, 1992, by A. Schiff, et al.

A.6 1987 WHITTIER NARROWS, CALIFORNIA EARTHQUAKE

Summary By M.J. O'Rourke

The Whittier Narrows earthquake (magnitude 6.0) occurred in the Los Angeles basin region at approximately 7:42 AM on October 1.

A.6.1 System Description

The affected region is generally suburban in character with moderate population density. It is served by roughly two dozen water utilities. The primary supply is groundwater pumped from the San Gabriel Valley Groundwater Basin. This supply is augmented with water purchased from the Metropolitan Water District of Southern California, a regional water wholesaler. The system consists of the following components: distribution pipe, groundwater wells with pumps, booster pump plants, storage tanks and reservoirs. The distribution system in the City of Whittier consists of roughly 135 miles (225 km) of pipe, over 80% of which is cast iron.

The system has a storage capacity of about 25 million gallons (MG) and a normal demand of roughly 7 MG/day. Storage includes the Garvey reservoir, a 10 MG reservoir in Whittier, as well as a number of tanks. A partial inventory of tankage is: 1.5 MG pre-stressed wire-wrapped concrete tank (30' H, built 1962), 1.0 MG buried concrete tank (built 1921), three 0.5 MG tanks (one elevated steel, one 70'D, and 30'H).

A.6.2 System Performance Vis-a-Vis Fire Suppression

Seismic damage to distribution piping (i.e., water main breaks) resulted in reduced water pressure (50 psi vs. normal pressure of 80 to 100 psi) for a period of two days following the earthquake. In addition, limited areas were without water for a period of hours. Fire service was informed of the reduced pressure but, reportedly it did not cause problems. As a precaution a 5,000 gallon tanker, normally used for grassland fires, was relocated as a reserve, but was not needed.

A.6.3 Component Performance

Available information on component performance and associated impacts on water supply/fire fighting capability are summarized:

- **Distribution Piping** - Damage rate of roughly 0.08 repairs per kilometer resulted in reduced water pressure and limited, short term, outages.

- **Wells and Pumps** - Mechanical equipment, wells and well casings were undamaged. However, commercial power was lost at a number of wells and pump stations, in one case for eight hours. In some instances back-up power generators were in place.

- **Storage Tanks** - A number of tanks suffered repairable seismic damage, but apparently none collapsed. Reported damage included circumferential cracking in wire-wrapped tank, diagonal cracking in a buried concrete tank, and broken tension struts in an elevated steel tank. In addition, several steel tanks experienced damage to inlet, outlet, and drain pipes. In the case of a fractured drain pipe, the tank was emptied into the system and the pipe repaired.

- **Reservoirs** - There was no reported damage to reservoirs.

A.6.4 References

The information presented above is based upon "The Whittier Narrows, California Earthquake of October 1, 1987 - Response of Lifelines and Their Effects on Emergency Response" by A. Schiff, *Earthquake Spectra*, v. 4, n.2, p 339-361, 1988.

A.7 1923 KANTO, JAPAN EARTHQUAKE

By C. Scawthorn

This section summarizes effects of the 1923 Kanto earthquake on the components of the water supply systems for Tokyo and nearby cities. The 1923 earthquake was followed by the largest urban conflagration in history, in which approximately 140,000 persons perished. The earthquake and conflagration are first briefly summarized, followed by a discussion of the effects on water supply components, and a qualitative analysis of the contribution of the water supply damage to the occurrence of the conflagration.

A.7.1 Seismological and Overall Damage Aspects

The M 7.9 Kanto earthquake (Usami,1987) occurred at 11:58 AM local time Sept. 1, 1923, with an epicenter located just offshore in Sagami Bay, at 139.5 E, 35.1N, (see Figure A-7.1). Damage was extensive, with major crustal movements (maximum 2m. uplift), significant numbers of engineered buildings sustaining structural damage (Freeman. 1932), destroying approximately 128,000 houses and damaging another 126,000 (Kanai, 1983), with extensive liquefaction (Hamada et al, 1992) (see Figure A-7.2) and landsliding (ASCE, 1929). Shaking intensity has been estimated at JMA 6 (equivalent to MMI IX) (Kanai, 1983), (see Figure A-7.3). A major tsunami (4~6 m. in height) affected the Miura and Boso peninsulas, destroying 868 houses (Kanai, 1983; Hamada et al 1992).

Tokyo had long been recognized as a major conflagration hazard, due to a dense urban aggregation of wood buildings. Due to a recent dry period and nearby typhoon, meteorological conditions were particularly adverse at the time of the earthquake, with hot (approximately 26 C, 80 F) dry winds of approximately 12.5 meters per second (28 mph) at the time of the earthquake. Winds grew continuously all day, reaching a maximum of 21 meters per second (48 mph) at 11 pm that evening. The result was a major conflagration which burned for several days, causing approximately 140,000 deaths and destroying approximately 447,000 houses by fire (Kanai, 1983; Hamada et al 1992).

A.7.2 Damage to Water Supply Components

Water supply systems for the cities of Tokyo, Yokohama, Kawasaki, Yokosuka were all generally similar in configuration, drawing their supply from rivers emanating from the mountains surrounding the Kanto plain, conveying the water by gravity to terminal stilling basins via concrete or cast iron aqueducts, and thence via distribution systems composed mainly of cast iron pipe. The performance of each component (ASCE, 1929) is presented below.

Aqueducts: Tokyo aqueducts drew water from the Tama River, about 22.5 miles distant from the city, and consisted of a concrete horseshoe tunnel 11 ft. in diameter, approximately 5.5 miles in length; a concrete-lined cut and cover tunnel 7.5 ft. in diameter; a cement lined cast iron pipe 6.5 miles in length, and a concrete pipe 15 miles long (diameters not available for the latter pipes, but of 100 and 125 cfs capacity, respectively). Damage to all of these conduits was nil to minor.

Yokohama sources consisted mainly of intakes on the Michisi and Kamino rivers, about 29 miles distant from the city. Both sources were buried by landslides. Conveyance was entirely by metallic pipe in cut-and-cover sections, several tunnels and crossing numerous bridges. Pipes consisted of three cast iron (CI) (varying in diameter from 15.5 inches to 38 inches), and one of welded steel (42, reducing to 36, inches). The smaller CI conduits were "somewhat" damaged, while the larger CI and steel pipe damage was negligible. Joints however had to be recaulked, and the steel pipe riveted joints worked loose.

Kawasaki takes water from the Tama river via short sections of 24 and 18 inch concrete pipe, and then via a 4.3 mile long 12 inch wood pipe. No damage was reported, except where the wood pipe was exposed by earth slides.

Yokosuka's conduit was a 20 inch CI pipe 31.7 miles in length, laid in tunnels, hillside benches, embankments over valleys, and over bridges. For 8.5 miles from the intake the pipe was laid in good ground and there was practically no damage. The central section 17.1 miles long had many embankments across valleys and these were all badly damaged and required rebuilding. In high embankments the pipes were laid on concrete piers which were overturned or inclined, resulting in the pipes being thrown off the piers, in some cases as much as 30 ft. from the center line (ASCE, 1929). Pipes crossing bridges were undamaged except where abutments or approach embankments settled, pulling pipe apart.

In summary, intakes at rivers appeared to susceptible to landslides, and large conduits performed well except in poor ground or at places of large differential settlements, such as bridge abutments.

Settling Basins: Tokyo's Wadabori settling basin, roofed over and of reinforced concrete, was substantially damaged, with hinges forming at the top and bottoms of columns. The Sakai settling basin suffered cracked lining, broken walls and similar damage.

Yokohama had a settling basin near its source, consisting of 4 ponds each 230 ft. by 175 ft, 16 ft. deep, with 18 inch floors and concrete gravity walls 3.5 ft. thick at the top. Damage was minor. Nearer the city, three settling basins existed: the Nishitani basin consisted of 8 ponds each 212 ft. by 151 ft. by 8 ft. deep, of concrete. Damage was minor. The Kawai settling basin was located on a mountain side, and was put out of use, with brick division walls being destroyed. The Nogeyama settling basin was the first of its kind in Japan, being constructed in 1885-97, primarily of unreinforced masonry. It was completely destroyed, and abandoned - the primary mode of damage seems to have been wall failures.

Kawasaki's settling and filtration plant was of concrete construction, with the floor cast over a one foot bed of clay, and the walls founded on small wooden piles with crushed rock fill. The settling and distribution ponds were badly cracked. Interestingly, a steel elevated water tank, supported by laced channel columns with rod diagonals, was undamaged.

Yokosuka's filtration beds and settling basins were on firm ground, and were generally undamaged, with only minor cracking.

In summary, basins of reinforced concrete performed well when on firm ground.

Distribution System: Tokyo's distribution system totaled 723 miles in length, and included 255,000 lineal feet of 16 to 60 inch diameter cast iron (CI) trunk line pipe, and secondary mains from 4 inch to 14 inch, totaling about 700,000 lineal feet. No specific information is available regarding damage to the Tokyo distribution system, except that cast iron pipe throughout Tokyo suffered in places, especially in filled ground and along river banks.

Yokohama's distribution system totaled 170 miles of CI pipe, ranging in size from 4 inch to 36 inch. All pipe trenches were re-excavated in order to recaulk joints - in general, smaller pipe was more damaged than larger, and fittings, tees and elbows were the worst damaged. Wherever the pipe crossed bridges, pipes were broken. There were 83,600 services to houses - 80% of these houses were burned, and their services were destroyed, while services to the 20% unburned were undamaged.

Yokosuka's distribution system comprised 117,000 lf, about 50% of which was on reclaimed land. About 50% of joints required recaulking - damage was greatest to smaller diameter pipe, and to fittings etc, repeating the experience in Yokohama, as did the experience with burned houses also having services destroyed.

In summary, distribution systems were constructed of cast iron, and sustained substantial damage, mostly in smaller diameters, and at tees, elbows and other fittings, and mostly in softer ground. A major impact was the additional damage to the distribution system as the fire grew in size, due to damage to building services, as the buildings burned.

Wells: While not a major source of potable water, numerous wells existed in the Kanto region, for industrial and other purposes. Of 96 artesian wells in the region employed for railway water supply, only 26 were affected, and many of these were not fully cased in metal. It was concluded that the damage to wells was rather light when compared with buildings and other structures.

A.7.3 Impacts of Water Supply Damage on Firefighting:

Analyses are not available as to whether Tokyo and Yokohama could have defended themselves against conflagration if meteorological conditions had been more favorable. In the actual event, conditions were extremely unfavorable - perhaps several hundreds ignitions occurred almost immediately, due to the lunch hour timing of the earthquake, at which time thousands of small grills were being employed. The ignitions were fanned by high winds, and grew rapidly in the densely built up neighborhoods of almost exclusively wooden buildings, which had been made more flammable by a recent dry period.

Tokyo and Yokohama, particularly the areas most heavily burnt, are low-lying. Damage to the distribution systems in these areas was heaviest, so that hydrants were probably often dry. Another important factor was the impact on <u>both</u> demand <u>and</u> capacity, of the fire. That is, as the fire grew, it impacted the water system in two mutually exacerbating ways:

- **Demand**: In general, fire growth is exponential, for a plentiful fuel supply. While the perimeter which must be defended will grow as the square root of the area of the fire, in general the net result is that the firefighting water demand increase exponentially over time.

- **Capacity**: As the fire grows in area, relatively more of the distribution system is available for supply, so that it would be expected that water supply increases. However, buildings within the fire are collapsing, breaking their services. In general, the net result is that the capacity of the distribution system is actually decreasing as the fire grows, due to the increased drain on the system due to hundreds or thousands of broken services.

This situation is typical of urban conflagrations, and undoubtedly existed in Tokyo, irrespective of the initial seismic damage to the distribution system.

On the other hand, the portions of Tokyo and Yokohama most heavily burnt are low-lying, with numerous canals and access to waterways. Therefore, secondary emergency water supply should have been available to the fire fighters. However, it is likely the equipment of the time was limited in capacity, and could not furnish the volume of water required to contain the large fires which quickly developed.

Therefore, the primary factor leading to the conflagrations in Tokyo and Yokohama was not the seismic damage to the water supply system, but rather the rapid growth of numerous simultaneous ignitions under a situation of adverse meteorological and dense wood building conditions. These conditions would have overwhelmed the fire service, and an undamaged distribution system, even had there been no earthquake. As in San Francisco in 1906, the city was waiting to burn.

A.7.4 Summary:

The 1923 Kanto earthquake resulted in strong shaking and widespread permanent ground deformations. In general, however, this did not severely damage the main water supplies to Tokyo and neighboring cities. The distribution systems, on the other hand, sustained numerous breaks, primarily in smaller pipes in low-lying ground. Notwithstanding the failure of the distribution system, however, the conflagrations which developed, and which were the main agent of damage in this catastrophe, were primarily due to non-seismic factors. These included a conflagration-prone built environment, and an extremely adverse ignition scenario and meteorological conditions. Under these circumstances, it is likely the fire service would have proved inadequate, even had there been no earthquake.

A.7.5 References

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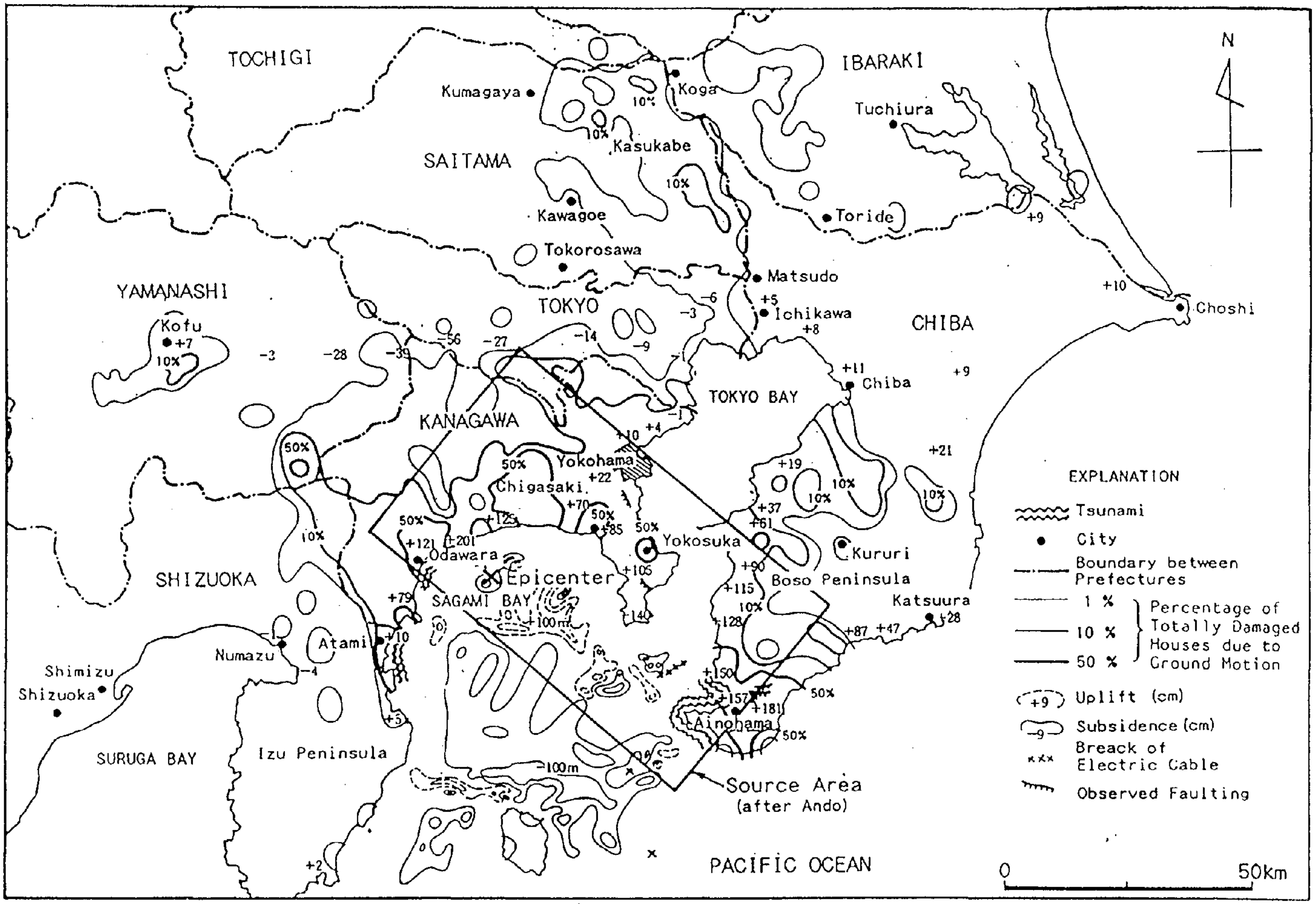

Figure A-7.1 - Map of Kanto and Chubu Districts Showing Distribution of Seismic Damage Caused by the 1923 Kanto Earthquake (from Hamada et al, 1992, as modified by Usami, 1987).

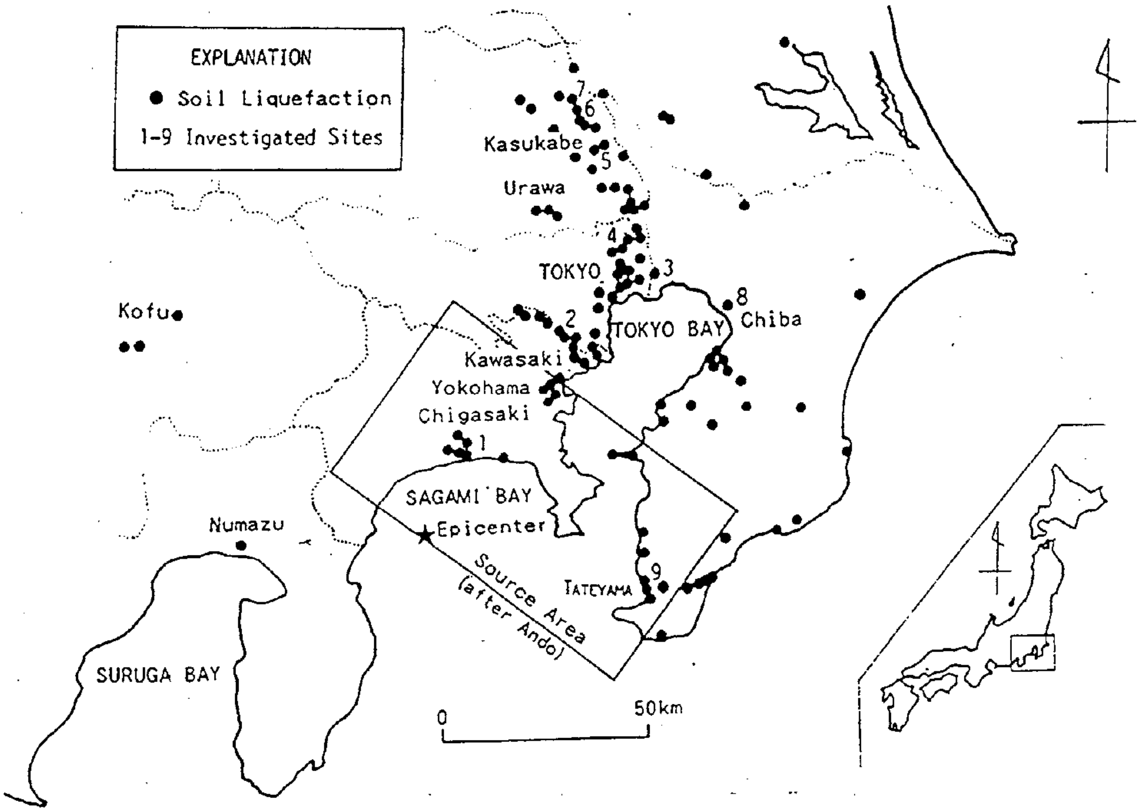

Figure A-7.2 - Map of Kanto and Chubu Districts Showing Principal Areas Affected by Liquefaction during the 1923 Kanto Earthquake (from Hamada et al 1992).

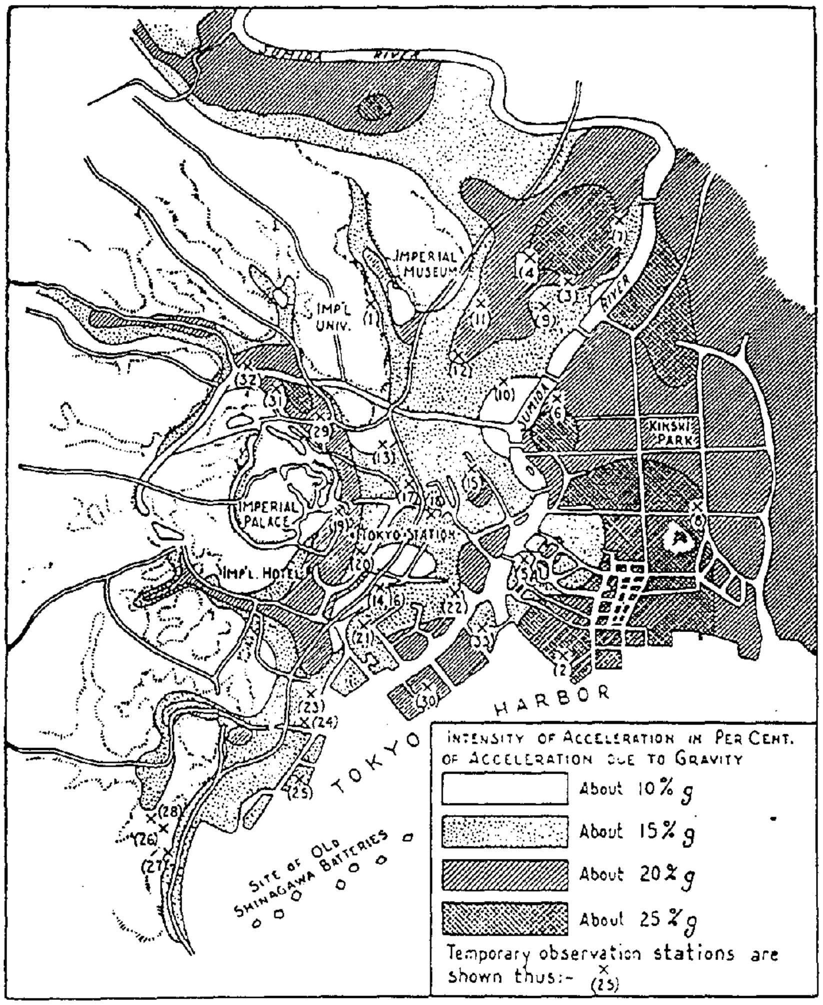

Figure A-7.3 - Distribution of Earthquake Intensity in Tokyo, September 1, 1923 (from Freeman 1932).

A.8 1906 SAN FRANCISCO, CALIFORNIA EARTHQUAKE

By C. Scawthorn and T.D. O'Rourke

This section summarizes effects of the April 18, 1906 San Francisco earthquake (magnitude 7.7) on the water supply of the City of San Francisco. The 1906 earthquake, which occurred at 5:12 AM, was followed by the largest urban conflagration in history to that time (only exceeded since, in peacetime, by the fire following the 1923 Tokyo earthquake). Fire-related aspects of the 1906 event are discussed elsewhere (Scawthorn and O'Rourke, 1989).

A.8.1 Water System Description

In 1906 water to San Francisco was supplied from two series of reservoirs. South of San Francisco, water was impounded by earth and concrete dams to form the San Andreas, Crystal Springs, and Pilarcitos Reservoirs. Transmission pipelines conveyed water from these reservoirs to a second series of smaller reservoirs within the city limits. Water then was distributed throughout the city by means of trunk and distribution pipelines. The San Francisco reservoir and pipeline network at the time of the 1906 earthquake bas been described by Schussler (1906) and more recently by O'Rourke, et al (1988). Reference is made to these works in presenting the overall configuration and operation of the system in this paper.

Figure A-8.1 presents a plan view of the 1906 San Francisco water supply adapted from maps prepared by Schussler. At the time of the earthquake, there was a combined volume of 88.7 billion liters in the San Andreas, Crystal Springs, and Pilarcitos Reservoirs. These reservoirs supplied nearly all water for the City of San Francisco in 1906, whereas today they represent approximately one-half of the local storage capacity in the San Francisco Bay Area. Transmission pipelines conveying water from the southern reservoirs were built mainly of wrought iron.

Within the city limits, there were approximately 711 km of distribution piping at the time of the earthquake, of which roughly 18.5 and 66.5 km were wrought and cast iron trunk lines, respectively. These lines were larger than or equal to 100 mm in diameter. The bulk of the system had been constructed during the years of 1870 to 1906.

A.8.2 Earthquake Performance of Pipeline System

Superimposed on Figure A-8.1 are the approximate locations of transmission pipeline damage caused by the earthquake. Flow from all transmission pipelines stopped shortly after the earthquake. Because telephone service was out, emergency control information had to be obtained by dispatching personnel into the field where maintenance crews reported on the damage.

Right lateral strike-slip movement along the San Andreas fault ruptured a 750 mm diameter wrought iron pipeline conveying water from the Pilarcitos to Lake Honda Reservoir. Over 29 breaks were reported north of the San Andreas Reservoir, where the pipeline was constructed parallel to the San Andreas fault. Fault movement near the San Andreas Reservoir was measured as 3.6 to 5.6 m (Lawson, 1908). Pipeline ruptures were caused by tensile and compressive deformation of the line. Over three months were required to reconstruct the pipeline.

Within 16 hours after the earthquake, repairs were made to that part of the Pilarcitos Conduit which was located within the city limits. Water then was pumped from Lake Merced through the Pilarcitos line into Lake Honda at a rate of approximately 25 million liters per day. Dynamic distortion of bridges was responsible for rupturing a 925 mm diameter wrought iron pipeline conveying water from the San Andreas to College Hill Reservoir, and for rupturing at three swamp crossings an 1100 mm diameter wrought iron pipeline conveying water from the Crystal Springs to University Mound Reservoir. The wooden trestle bridges all were damaged by strong ground shaking, with no damage or misalignment observed in their timber pile foundations. Approximately three days were required to repair the 925 mm diameter pipe, and over a month was required to restore the 1100 mm diameter Crystal Springs Pipeline.

Figure A-8.2 is a map of the 1906 water supply within the San Francisco City limits. Table A-8.1 summarizes the city reservoirs, capacities, and elevations above mean sea level. There were nine reservoirs and storage tanks, for a total capacity of 354 million liters. Approximately 92% of Ibis total, or 325 million liters, were contained in the Lake Honda, College Hill, and University Mound Reservoirs. These reservoirs and the pipelines linking them with various parts of the city were the backbone of fire protection.

All trunk lines, 400 mm or larger in diameter, are plotted in Figure A-8.2. These pipelines were plotted on the basis of a careful review and transcription of the oldest extant (1912) pipeline maps in San Francisco, which were provided by the San Francisco Water Distribution Department and correlated with reservoir and pressure district maps published by Schussler. Trunk lines are shown connected to the Lake Honda, College Hill, University Mound, Francisco Street, and Clay Street Reservoirs; all other reservoirs were connected to piping 300 mm or less in diameter. As listed in Table A-8.1, the University Mound and Francisco Street Reservoirs are at approximately the same elevation. The Francisco Street Reservoir was filled by water from the University Mound Reservoir, which then was pumped to Presidio Heights. Accordingly, the supply for Presidio Heights and Francisco Street depended on the pipelines from University Mound.

Superimposed on the Figure A-8.2 are the zones of lateral spreading caused by soil liquefaction, as delineated by Youd and Hoose (1978). Breaks in the pipeline trunk system crossing these zones are plotted from records provided by Schussler (1906) and Manson (1908). It can be seen that multiple ruptures of the pipeline trunk systems from the College Hill and University Mound Reservoirs occurred in the zones of large ground deformation, thereby cutting off supply of over 56% of the total stored water to the Mission and downtown districts of San Francisco.

Two pipelines, 400 and 500 mm in diameter were broken by liquefaction-induced lateral spreading and settlement across Valencia Street north of the College Hill Reservoir. These broken pipes emptied the reservoir of 53 million liters, thereby depriving fire fighters of water for the burning Mission District of San Francisco. As indicated in previous studies (O'Rourke and Lane, 1986), this local deformation ranks as one of the most devastating events of the 1906 earthquake.

A.8.3 Impacts of Water Supply Damage on Firefighting

As a result of the earthquake, over fifty ignitions occurred within a relatively short time, and quickly developed in several conflagrations, which burned for three days (Scawthorn and O'Rourke, 1989). Figure A-8.3 shows a map of the San Francisco water supply and the burnt area. All trunk lines of the College Hill and University Mound Reservoirs downstream of the pipeline ruptures are removed from Figure A-8.3 to show the impact and lack of hydraulic conductivity caused by severing these conduits.

With the College Hill and University Mound Reservoirs cut off, only the Clay Street Tank and the Lombard and Francisco Street Reservoirs were within the zone of most intense fire, and therefore capable of providing water directly to fight the blaze. The combined capacity of these reservoirs was only 21 million liters, or 6% of the system capacity. The usefulness of such limited supply was further diminished by breaks in service connections, caused by burning and collapsing buildings. Schussler identifies service line breaks as a major source of lost pressure and water. There were roughly 23,200 breaks in service lines, between 15 and 100 mm in diameter, (Figure A-8.4). Fallen rubble and collapsed structures often prevented firemen from closing valves on distribution mains to diminish water and pressure losses in areas of broken mains and services.

As is evident in Figure A-8.3, the Lake Honda Reservoir was able to provide a continuous supply of water to the western portion of the city. The fire eventually was stopped along a line roughly parallel to Van Ness Avenue, a wide street where water still was available from the Lake Honda Reservoir. Moreover, the southern and southeastern extent of the fire is bounded by areas south and southeast of the trunk system ruptures. It is likely that these unburned areas had water from the University Mound Reservoir. Figure A-8.5 presents a bar graph showing the reservoir storage in San Francisco as a function of time after the earthquake. The amounts of water corresponding to day 0 represent the quantities available roughly two hours after the earthquake struck. After four days, less than one tenth of the initial capacity of the College Hill, University Mound, and Lake Honda Reservoirs still was available. Two factors were critically important in preserving flow. Sixteen hours after the earthquake, water was pumped from Lake Merced into the Pilarcitos Conduit to supply Lake Honda. This action provided an additional 25 million liters/day, thereby maintaining capacity in Lake Honda for distribution to the western parts of the city. After repairs of the San Andreas Conduit over three days, approximately 30 million liters/day were conveyed to the College Hill Reservoir for distribution in the South Mission area of the city. By the fourth day, approximately 55 million liters of water were flowing into the city, in addition to the 25 million liters still available in the reservoirs.

A.8.4 Discussion

Loss of the Pilarcitos, San Andreas, and Crystal Springs Conduits in 1906 provides a graphic example of the vulnerable nature of transmission pipeline supply, and emphasizes the extreme importance of reliable emergency water supply close to users. System redundancy played a critical role in water delivery after the 1906 earthquake. The ability to disengage the Pilarcitos conduit from the ruptured transmission system south of the city and connect it with Lake Merced was of vital significance. Water to Lake Honda was provided in continuous supply to the western districts of San Francisco where the fire was stopped. The lack of redundancy across zones of large ground deformation in the Mission district meant that pipeline failures resulted in the loss of over 56% of the city supply. Only 6% of the total reservoir capacity was directly accessible at the time of the earthquake to fight the fires in downtown San Francisco.

While potable water needs are very elastic, water for firefighting constitutes a large real-time demand which however can be obtained from a variety of non-potable sources. It is a very important lesson that the fire developed in San Francisco, when it is surrounded on three sides by the largest body of water on earth. In order to assure a reliable supply for the future, San Francisco subsequently constructed an Auxiliary Water Supply System (AWSS) for fire fighting purposes, consisting of approximately 200 km of extra-heavy seismically detailed pipeline, a storage capacity of 43 million liters, 175 additional cisterns (each with about 300,000 liters equivalent to one hour fire engine supply), Pump Stations 1 and 2 to draw water from the Bay (each with about 660 liter/sec high pressure pumping capacity) and additional connections for San Francisco's two fireboats to also pump saltwater into the AWSS (each fireboat approximate capacity also 660 liter/sec). More recently, the San Francisco Fire Department has developed the Portable Water Supply System (PWSS), which offers additional and very flexible capability to convey water to those major portions of the City outside the areas protected by the AWSS. Combined, these systems represent the largest and most seismically resistant emergency water supply system in existence.

A.8.5 References

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TABLE A-8.1 Summary of Reservoirs and Water Capacity at Time of Earthquake

Capacity	Reservoir (million liters)	Elevation (m)
University Mound	140.0	50.3
Francisco Street	11.4	41.1
College Hill	53.0	77.7
Lake Honda	124.9	111.3
Lombard Street	9.5	93.0
Potrero Heights	3.0	96.0
Presidio Heights	2.6	122.U
Clav Street Tank	0.9	114.3
Clarendon Heights	1.9	182.9

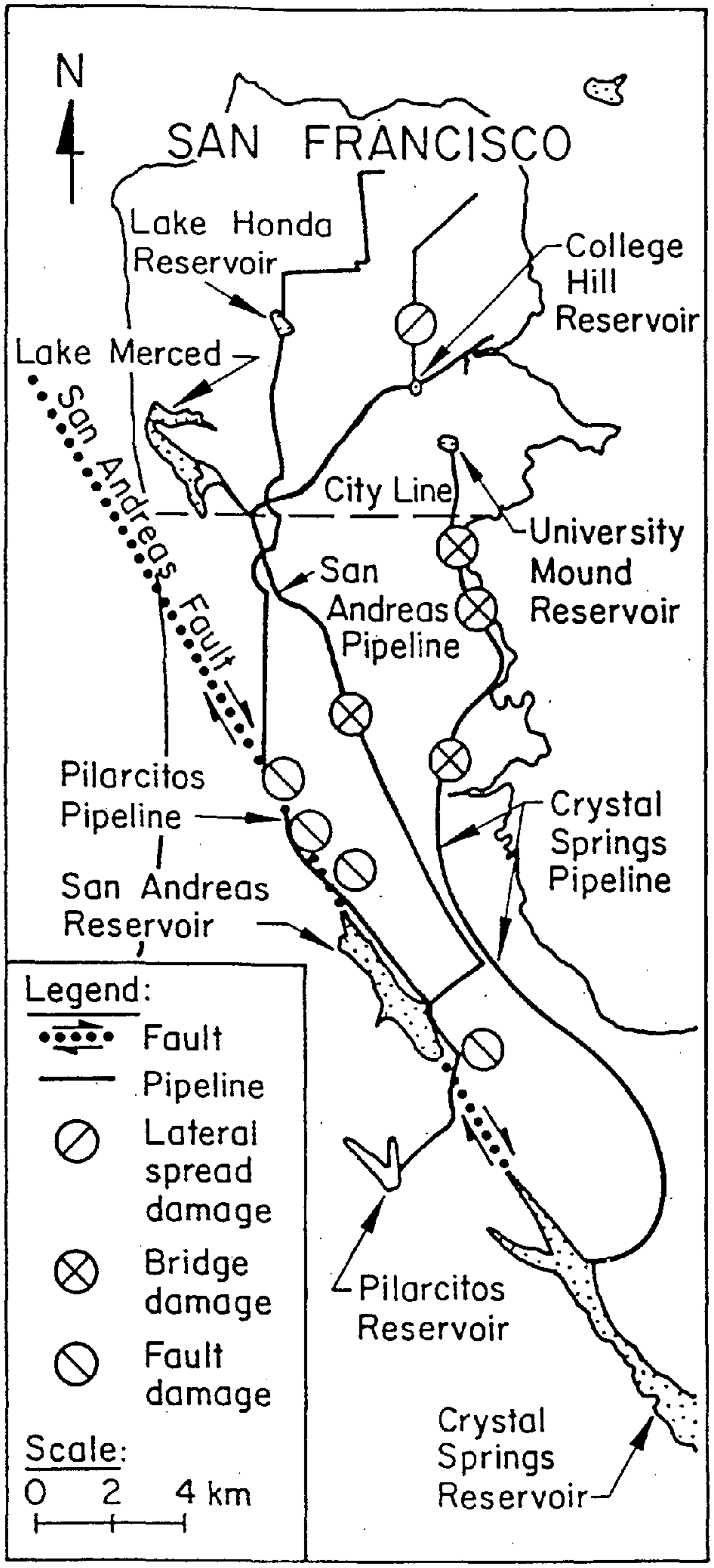

FIGURE A-8.1 Approximate Locations of Transmission Pipeline Damage, 1906 San Francisco Earthquake

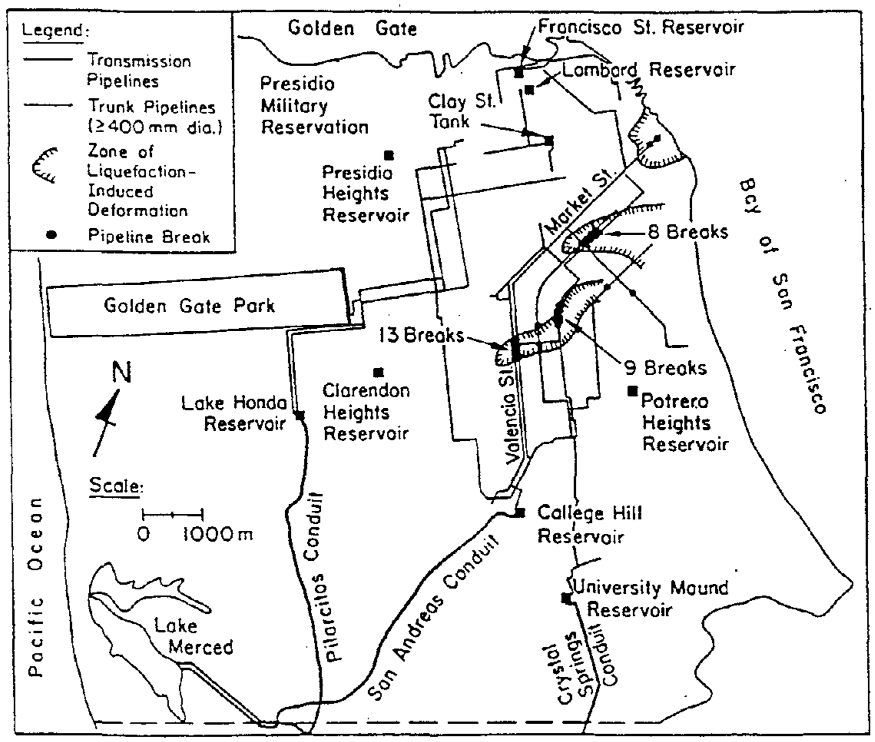

FIGURE A-8.2 1906 Water Supply Within San Francisco City Limits, Trunk Lines, 400 mm or Larger

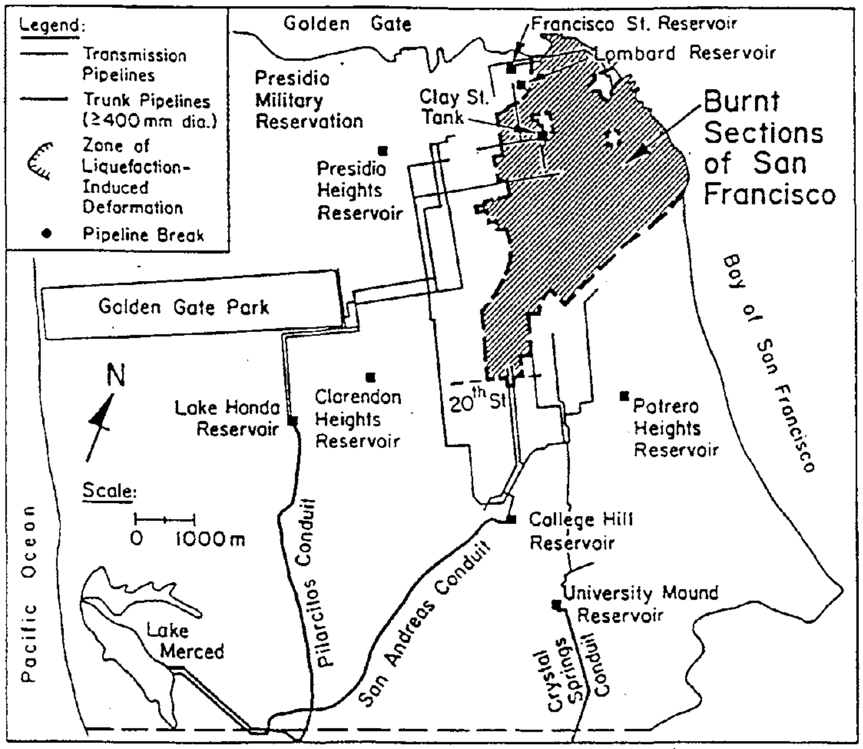

FIGURE A-8.3 San Francisco Water Supply and Area Burned During Earthquake

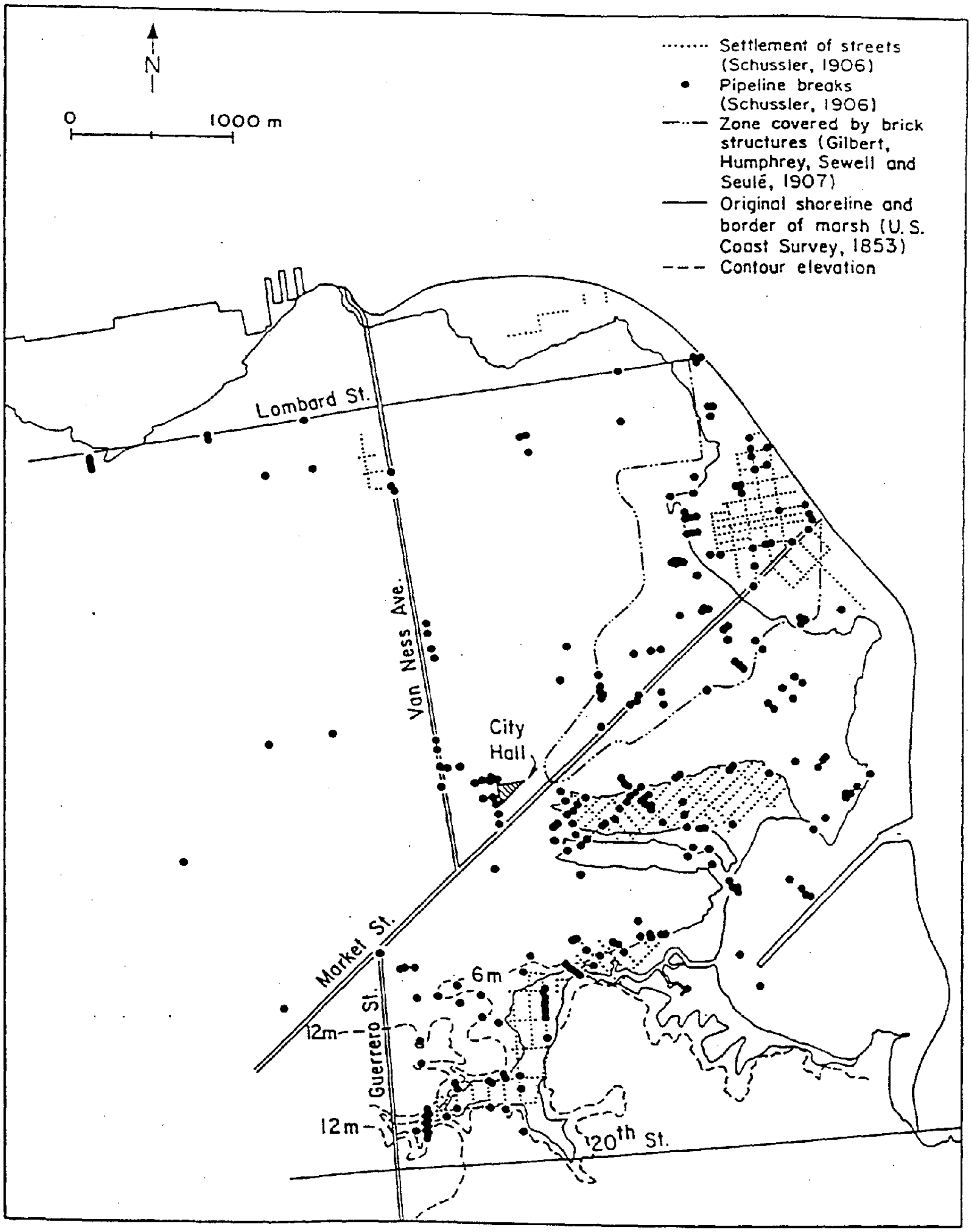

FIGURE A-8.4 Pipeline Breaks and Street Settlements in San Francisco after 1906 Earthquake (after Schussler, 1906; Manson, 1906)

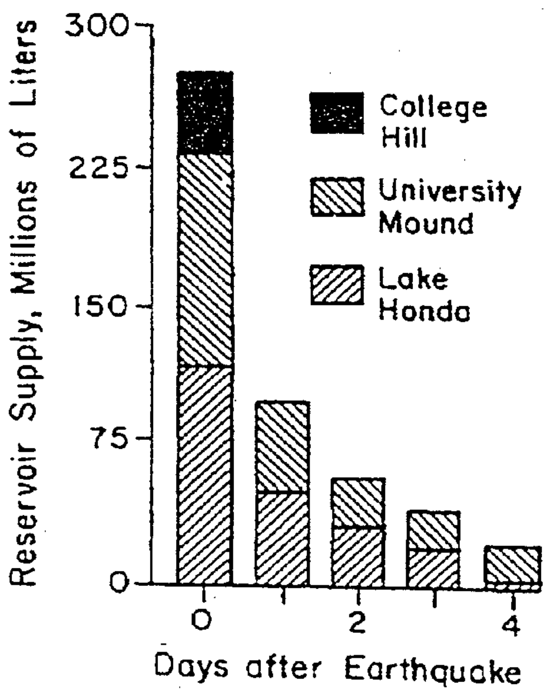

FIGURE A-8.5 Reservoir Storage in San Francisco, as a Function of Time after the Earthquake

A.9 1993 DES MOINES, IOWA FLOOD

By D. Ballantyne

A.9.1 Introduction

The City of Des Moines is located at the confluence of the Des Moines and Racoon rivers. On Sunday, July 11, 1993, the Des Moines, Iowa, Water Works 40 mgd water treatment plant, serving 250,000 people, was submerged by flood waters from the Raccoon River, leaving Des Moines without drinking water and water for fire suppression. Floods had previously struck the Des Moines area in 1947, and 1986. A 25-foot levee had been constructed around the treatment plant to keep out future flood waters. The model used to predict flood levels had not consider backwater effects, as historically, the Racoon and Des Moines rivers had not flooded at the same time. When the levee was constructed, the Water Works had suggested building the levee to a level of 29 feet, but was overridden by the Corps of Engineers. The 25 foot levee was over-topped in 1993 with water rising to a maximum of 26.75 feet. Normal flows in the river range from 300 to 400 cfs; flows during the 1993 flood reached 68,000 cfs. Subsequent to the 1993 flood, a new levee has been built to 31 feet.

During the same period, much of the downtown area lost electrical power service due to flooding of substations. Downtown Des Moines was closed for a week , the main cause reported was loss of water supply, and not loss of electrical power.

The city's largest employer is the Principal Insurance Company which owned a 40 story office building, the largest buidling in town, as well as 4 additonal buildings totaling 1.2 million square feet of real estate. Their business operations were severely curtailed as result of loss of water for cooling and fire suppression.

The Des Moines flood experience is described in this document because it is a rare example of complete failure of a water system for an extended period, similar to what has occurred in past, and is expected in future earthquakes.

The flood waters receded within two days, the plant site was dewatered, and the first finished water pump motor was pulled for reconditioning. The treatment plant was back in service by July 16, and water was being pumped into the system the following day, seven days following the submergence.

The distribution system was filled and pressurized within 12 days, and water quality restored for drinking (boil water order removed) 19 days after the treatment plant was flooded. Final restrictions were removed 29 days after the initial flooding.

There was a total of $12 million damage, $10 million of which was covered by insurance. FEMA and the state reimbursed them for most of the balance. There was an additional loss of $2 million in revenues, and $6 million was diverted from capital improvements to flood recovery.

A.9.2 Power and Communication

The two power feeds to the plant were submerged. The treatment plant had two 4 MW diesel generators, the largest in North America. Both were rendered useless after there controls flooded. The Corp of Engineers supplied 2, 2 MW units that were used three times during the period the power system was being restored. These generators drove a 1750 HP, 30 mgd finished water pump.

Motors 2 HP and smaller were replaced. Larger motors were steam cleaned, baked, bearings replaced, rewound, and varnished. A benefit of the flood is that all the replaced/refurbished motors are expected to last for 20 years.

The phone equipment, and therefore communications and telemetry, at the plant was lost. The new communications systems is radio, and is installed on the second floor. The radio system is used for both voice and data.

A.9.3 Water Quality

Approximately 1,000 miles of distribution piping was drained, and there was a concern that contaminated water had backflowed into the system. While the system was being refilled, chlorine was fed at a rate of 6 mg/l, too high for consumption. The resulting chlorine odor kept people from drinking the water, as well as help disinfect the system. They started to bleed the system at hydrants.

Water quality tests were taken at 140 locations; 18 of them failed. It was determined that all the failed samples had been taken by an individual inexperienced in collecting samples. On retest, all passed. There was only one instance where there was back-siphonage from a toilet bowl, resulting in green water.

A.9.4 Public Health Issues

Tank trucks hauling 1,500 gallons of potable water were brought to 17 distribution sites within eight hours. The Health Department contacted the Iowa Motor Truck Association for assistance in getting tankers, and got more than they knew what to do with. They ended up serving a total of 104 sites, each with portable tanks which were regularly filled. There was a need for extra spigots to distribute water.

The Health Department was concerned about lack of water for food preparation at restaurants. They imposed strict guidelines and required simple menus to minimize water requirements. Disposable plates and utensils were recommended. Only one restaurant was closed out of 1,500 inspected.

The National Guard provided seven reverse osmosis units to treat water for area hospitals.

A.9.5 Fire Department Operations

The fire department moved tankers, normally used in outlying areas, to central locations for dispatch. Eighteen 7,000 gallon tankers were set up at nine fire stations to be used in conjunction with pumpers for the duration of the outage. The tankers came from milk and alcohol haulers. There was one major fire during the period where seven tankers delivered water on rotation. It took six hours to bring under control. A drafting point was set up at a firm water supply to refill the tankers.

Fire alarm and monitoring systems were inoperable due to loss of power. Typically there was no backup power. There were 225 alarms wired directly to the fire department.

Usage of buildings above the third floor was generally not allowed. One eleven story retirement home was allowed continued occupation, but was required to have a fire watch on each floor to watch and smell for fire/burning material.

The fire department practices drafting water from the river every year. They have ten sets of 1,000-foot long, 5-inch hose with strainers.

A.9.6 Telephone Company Impact

US West came within a few minutes of loosing service due to the disrupted water supply, used for equipment cooling. Ultimately, backup pumps and generators were used to pump chiller make-up water from tank trucks to the 15 floors of the facility during the outage. They have installed onsite wells to avert future outages.

A.9.7 Business Interruption

In general, it was found that businesses take care of themselves. The Principal Insurance Company brought in their own water tank trucks to operate the air conditioners, and emergency generators to provide electrical power for the buildings. Early in the emergency, they installed stand-alone air conditioners to cool computers. They rented KYBOS, portable toilets, for staff use. They were limited to using only the four lower floors of their building, while staffing it to monitor for fires. The building's 1,200 workers were limited to four hour shifts, which cost the company about $0.5 million per day. The cost to Principal, excluding loss of labor, was an estimated $2 million.

The Monferd Beef Company had purchased a one-half million dollar portable water treatment plant to enable them to use water from the river in the event of an emergency. They used it.

A soft drink bottler moved their production to another facility.

During the time the water system was not functioning, hotels were limited to only using the 3 lowest floors due to lack of fire fighting capability in the higher floors.

A.9.8 Mitigation for Future Events

The water department concluded they did too good of a job. After the recovery, they proposed construction of a new water treatment plant to ensure future water supplies in the event of a disaster. People generally were opposed to the idea because such events only occur every 50 to 100 years.

A.9.9 Conclusion

While flooding damage costs to the Des Moines Water Works were for the most part reimbursed. The cost of loss of service, business interruption, would have been covered by business interruption insurance if they had coverage, or taken as a loss. In retrospect, the cost of construction of a higher dike system should have been compared with probable losses, times the probability of those losses occurring, and a decision made based on the cost/benefit of the mitigation effort. Unfortunately, it appears that the decision not to construct a higher dike system was based on uncertain data about maximum flood levels that proved to be inaccurate.

A.9.10 References

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Morgan, Charles; Chief, Des Moines Fire Department

A.10 1991 OAKLAND HILLS, CALIFORNIA FIRESTORM

By J. Eidinger

A fire occurred on Saturday, October 19, 1991, north of the Caldecott Tunnel near the eastern end of Buckingham Boulevard (Olson et al, 1992). This three-acre fire was controlled Saturday. Fire fighters, who had doused the burn area with water before leaving the fire scene before leaving the scene Saturday evening, returned Sunday morning to continue mop-up operations. Weather conditions had changed dramatically overnight and were extreme on Sunday October 20. A combination of high temperatures, very low humidity, and strong, hot, dry Foehn or Diablo winds from the northeast produced extreme fire hazard conditions. Fire fighters were dealing successfully with a few minor flare-ups when, at about 10:45 am, a spark was blown by gusting winds to a fuel-rich spot just outside Saturday's burn area. Within a few minutes, the fire was out of control and spreading very rapidly. By 11:15 am the full perimeter of the previous day's burn area was involved and the fire had ignited the shake roofs of nearby houses, The fire raced up canyon slopes towards Grizzly Peak Boulevard and was simultaneously pushed down slope along Buckingham Boulevard by the increasing winds. Within 15 minutes, by 11:30 am, the fire had advanced more than a mile southward into the Claremont area and crossed Highway 24 into the Rockridge area (see Figure A-10.1).

By the time the fire reached its full extent, more than 370 fire units and over 1,000 fire fighters fought the fire. The firestorm killed 25 people, injured 150, and burned over 1,600 acres. It destroyed 3,354 single family houses, 456 apartments, and resulted in about $1.5 billion in damages.

A.10.1 Water System Infrastructure

The oldest parts of the pipe distribution system within the firestorm area date back to the early 1900s. The oldest reservoir is the Dingee reservoir, placed in service in 1894. Age of pipelines was an important point in considering the performance of a water distribution system during the firestorm. Certain pipe materials, particularly unlined cast iron, have degraded performance over time. Over time, the inside surface of unlined cast iron pipe becomes rough. This process is called tuberculation. As the pipe surface becomes rougher, it takes more energy for a given amount of water to travel past a particular point. This energy loss is more pronounced in small diameter pipe. The net result is that a old cast iron pipe can often have a capacity less than half that of when it was originally installed.

A.10.2 Performance of the Water System During the October 20, 1991 Firestorm

A.10.2.1 Assumptions

In the following sections, the amount of water used in a particular pressure zone at a particular time during the fire was calculated by using available data from known pump rates and reservoir elevations. This data was available from a real time SCADA data acquisition system. This calculation tells how much water was used in the zone: it does not say who used the water, and for what purpose. For example, the water could have been used for normal residential purposes (drinking, irrigation), for fire fighting by fire departments, for civilian fire fighting (wetting down roofs with garden hoses), or lost through broken service connections, after a structure burned down.

A.10.2.2 Water System Response - North of Highway 24

In this section we present the results of the analyses for the areas north of Highway 24. This area is shown as the upper dotted circle in Figure A-10.1.

For the three hour period from 8:00 am to 11:00 am, October 20, 1991, water usage in this area ranged from 1,000 to 2,000 gpm. Between 11:00 and 12:00 noon, the amount of water used in the fire ignition area increased to about 1,000 gpm, reflecting initial use by the first responding fire fighters. By 12 noon, many more fire fighters arrived in the area, and the amount of water used in the area jumped up from about 2,500 gpm to over 12,000 gpm. Part of this water was being used by fire fighters to fight the fire. Part of this water was lost through broken service connections, after houses had burned down. For the two hour period from 12 noon to 2:00 pm, water usage averaged about 13,000 gpm.

Beginning at 2:00 pm, three of the water tanks in the area were emptied. Due to the drawdown of these reservoirs to empty, the total water used in the area dropped to about 8,000 gpm from 2:00 pm to 5:00 pm. By the time the water tanks were empty, essentially all structures that ultimately burned in the area had already been burnt.

By 5:00 pm, the spread of the firestorm reached its maximum extent. From 6:00 pm through the night and into October 21, water usage averaged about 6,000 gpm.

A.10.2.3 Power Outages, Emergency Equipment and Empty Reservoirs North of Highway 24

Power failures in PG&E's system temporarily shutdown five pumping plants that replenish the reservoirs. Based on analysis of actual pumping plant capacities, if no power outages had occurred, or sufficient on-line backup power had been immediately available, then the total water use in the firestorm area in the upper circle in Figure A-10.1, between 12 noon and 2:00 pm, could have been increased by 1,250 gpm, to somewhat over 14,000 gpm. It is doubtful if this moderate increase in available water, along with other factors (fire fighting teams, fuel load, weather), would have had appreciable effect on halting the spread of the fire at this time.

A.10.2.4 Effect of Failed Service Connections North of Highway 24

As the fire spread through the area, structures were burned to the ground. In many cases, when a structure burned down, the service connection piping from that structure into the water distribution main became exposed.

Beginning at about 3:00 pm on October 20, 1991, the water utility personnel began closing off the taps of the distribution mains to these failed service connections. This effort lasted until about 7:00 pm that day, when it became too dark to continue operations. The effort was restarted at about 8:00 am October 21, 1991. By about 3:00 pm October 21, 1991, water utility personnel had closed off over 2,000 service connections.

Under normal conditions, a single 5/8 inch service connection can provide 20 gpm to a house. Many houses in the firestorm area have even larger service connections (3/4 and 1 inch connections are not uncommon - larger connections, from 2 - 6 inches, supply a few customers, such as schools and apartments). During the firestorm, lesser amounts of water actually flowed through a open service connection, owing to competing demands for water from fire fighters and other open service connections. Also, not all service connections were likely to be completely open due to the destruction of the house.

As an estimate of water lost through failed service connections, abandoned flowing hydrants, and other causes, we estimate that about one-third of all water used during peak water usage times (12 noon to 3:00 pm, October 20, 1991) was lost through failed service connections. This one-third estimate is based upon allowance for loss of water used in developing insurance rating guidelines for required fire flows (National Board of Fire Underwriters, 1956).

A.10.2.5 Water System Response - South of Highway 24 and West of Highway 13

Due to the limitations of some of the old pipe distribution system in this area (lower left circle, Figure A-10.1), it is not surprising that fire fighters reported lack of water at some locations. For example, Captain F. Baleria reported that:

"... connected to hydrants at Country Club and Beechwood, and Country Club and Bowling, and had barely enough water to supply one 1 1/2" hose line."

Similarly, in another after action report, Captain G. A. Flom reported that:

"... laid a hose from the hydrant [at Rockridge Blvd. South]... ...crews extinguished an apartment house on the east side of Margarido ... when the water failed. ... at this time, crews were in full retreat. ... restored water pressure ... enabled us to set up portable hydrants on Margarido and attack and suppress the house fire, saving the structure near the original hydrant."

These reports show that there was intermittent supply of water in this area. This occurred because the pipeline network was largely made of pre-1930 4" pipe; and severe overdrafting on the system. Reservoir supply to this area was never compromised (minimum storage reservoir levels throughout the fire were at least 5,000,000 gallons in the Dingee and Estate reservoirs serving this area).

At one location supplementary water was delivered using 5" diameter hose (San Francisco's portable water system), but these hoses failed when vehicles drove over the hoses; reportedly, fire department staff were unable to prevent vehicles from interested parties from entering the fireground area. When available, this supplementary water contributed under 3% of the peak fire flows actually delivered.

A.10.2.6 Overall Water Use South of Highway 24

For the three hour period from 8:00 am to 12 noon, October 20, 1991, water usage in this area averaged around 3,500 gpm. This amount of water usage reflects the amount of water used by residents in this area, for normal uses, such as consumption, irrigation, etc.

Between 1:00 pm and 4:00 pm, the amount of water used increased to about 9,000 gpm to 12,000 gpm, reflecting initial fire fighting efforts in the area.

Beginning around 4:00 pm, water usage increased to over 16,000 gpm. Water usage ranged from 16,000 gpm to over 20,000 gpm for the next ten hours.

Water usage decreased to about 14,000 gpm, from 2:00 am October 21 to about 7:00 am. Water usage declined to about 4,000 gpm by 2:00 pm October 21.

A.10.2.7 Power Outages, Emergency Equipment, Empty Reservoirs South of Highway 24

Power failures in PG&E's system shutdown certain pumping plants in that replenish the reservoirs south of Highway 24. The Dingee Pumping plant lost power at about 12:13 pm, October 20. PG&E power was restored to the Dingee pumping plant at 5:46 pm October 20. If the Dingee pumping plant were not stopped due to the power outage, an additional 9,700 gpm flow would have been available to the B5A Dingee Zone. As the reservoirs that serve the Dingee zone were never emptied, this additional water would not have had much impact on increasing available fire flows in the area. No after action fire reports were obtained that suggested that there was a lack of water in the first lift served by this pumping plant. Instead, pipe size limitations effectively precluded use of higher flow rates in the area. This suggests that PG&E power outages at the main pumping plant serving this area had *no appreciable affect* on the firestorm outcome.

A.10.2.8 Effect of Failed Service Connections South of Highway 24

As the fire spread through the area, structures were burned to the ground. In many cases, when a structure burned down, the service connection piping from that structure, into the water distribution main became exposed.

Beginning at about 8:00 am on October 21, 1991, water utility personnel began closing off the taps off the distribution mains to these failed service connections in this area. The effect of closing these service connections, beginning at about 8:00 am October 21, resulted in a decrease in water usage from about 11,000 gpm to about 5,000 gpm over the next four hours. This decrease is also likely due to the cessation of fire fighting efforts in some areas. From this trend, we conclude that about 4,000 - 6,000 gpm was being lost through failed service connections, at a more or less steady rate, from 5:00 pm October 20 through 8:00 am October 21. This rate of water loss is about one-quarter to one-third of the total water used in this area during this time interval.

A.10.3 Historical Fire Flow Guidelines for Water System in Firestorm Area

A.10.3.1 Pre-1929 Designs for Lower Elevation Areas

Part of the area covered by the October 20, 1991 firestorm was built prior to the formation of the present day water utility (pre-1929). The Dingee reservoir and local water distribution system was built by Mr. Dingee, in the 1890s. The standards that Mr. Dingee used to build this system are not known. No documented evidence could be found of a fire flow standard in use when the distribution pipes in the area were built.

A.10.3.2 Recent Designs for Higher Elevation Areas

Currently, the Oakland Fire Department has set 1,500 gpm for 2 hours as the required fire flow for new residential construction in the Oakland Hills area east of Highway 13. All construction post-1973 for these areas included a minimum fire flow of 1,500 gpm for 2 hours. Almost all areas east of Highway 24 complied with these fire flows at the time of the firestorm, including the two areas in Figure A-10.1 denoted by circles (top circle, right circle). Since over 90% of all structures in these areas were burned, it can be concluded that modern fire flow requirements provide insufficient flows when dealing with a large fire.

A.10.4 Conclusions

1. How well did the water utility respond to the firestorm emergency?

   - Based upon the water use analysis, over 90% of the areas within the firestorm had water systems which actually delivered at least the modern day fire flow requirement. Limited fire fighting operations in last 10% of the area resulted in less usage; however, the minimum required fire flow would have been provided, if fire fighters had needed it.

   - Due to high demands in the lower elevation old parts of the water system, water flow was interrupted to fire fighters located at some distances away from the pressure regulators feeding the area. For example, one fire fighting team drafted water essentially continuously throughout the day from the hydrant at Buena Vista Place, saving the houses on that street. The hydrant at Buena Vista Place is the first hydrant in the zone. Because of the small diameter, 1910 vintage cast iron pipe in this area, the distribution system could not simultaneously and continuously deliver water to hydrants further away from the pressure regulator, even though ample water was available in the reservoirs. In this respect, the water distribution system meets the intent of being able to fight a fire along an entire single block but was unable to deliver sufficient water to fight multiple fires in different locations.

2. How well prepared was the water utility to respond to the October 20, 1991 firestorm?

   - Based upon the fact that water utility's current design standard meet the requirements of the fire flows required by the Oakland Fire Department, and are consistent with those in use by other Water Agencies, and are consistent with guidelines used by the fire insurance business, the water utility was prepared to respond to normal fires.

   - Based upon the huge extent of the October 20, 1991 Firestorm, the water system ran out of water in certain areas, even in areas where the system completely meets today's fire flow standards. Therefore, we conclude that although the system was prepared for the fire it was designed to meet, it was not prepared to supply water for the October 20, 1991 firestorm. Substantial reservoir size increases, pumping plant size increases, and upgrades to 4 inch distribution piping would be required to supply water to many simultaneous fires within a single pressure zone. Such hardware improvements, well beyond that required by current standards, could have helped some fire fighting efforts - however, there may be other more cost effective approaches to improving total fire fighting capabilities, and reducing the risk of re-occurrence of this type of catastrophe.

3. Can the use of portable hose significantly reduce the impact of a firestorm?

   The Oakland Hills firestorm destroyed over 1,000 structures within 45 minutes. No significant deployment of portable hose can be accomplished in this time frame. Some use of this type of hose was done beginning 2-3 hours after the firestorm had spread. With more rapid and widespread deployment, ultimate fire spread boundaries could have been reduced by 10% to 30%.

4. Can the use of alternate water supplies be effective in a firestorm?

   As shown in Figure A-10.1, Lake Temescal was a possible source of water. However, air lift (helicopter or plane) access to this lake was exceedingly dangerous due to tremendous smoke and hilly terrain in this area. Further, the lake is several hundred feet lower than the locations of the houses, making access to this water supply difficult (long lengths of portable hose and many portable pumps required).

A.10.5 Alternative Strategies for Hill Area Firestorms

1. Replace Older Small Diameter Pipes with Larger Pipes. In certain areas of the system, severe overdrafting led to the inability to deliver water to hydrants where needed, even though there was plenty of water available in reservoirs. Replacement of smaller diameter pipelines will alleviate this constraint. However, this is a costly retrofit, and will provide only local benefits. The people that will benefit from this type of upgrade may need to be assessed the associated costs.

2. Modification of Today's Fire Flow Standards. During the course of the October 20, 1991 Firestorm, several of distribution reservoirs were emptied. The reason that these reservoirs were emptied was not that the reservoirs were incorrectly designed - they all met today's fire flow standard of 1,500 gpm for 2 hours. Rather, the prime reason that the reservoirs emptied was that the firestorm conflagration that occurred was far and beyond any size fire that was considered in design.

   One option is to consider increasing today's fire flow standards from 1,500 gpm for 2 hours, to some higher flow rate, and some longer duration. This choice is an expensive one to implement, especially in the existing built up areas of the water system service area. Further, increasing the size of reservoirs will adversely affect water quality.

3. Construction Upgrades. A significant contributor to the rapid spread of the fire was airborne shingles from roofs. If all houses in the area had clay tile or other non combustible roof, it is likely that the extent of the firestorm would have been considerably reduced.

   Cities rules could strictly enforce that new structures built in high risk fire areas be built with tile or similar roofs.

   It would also be highly desirable to rid the high fire risk areas of structures which already have shake roofs. This process can be mandated by City rules either at time of sale, at time of substantial structural improvements, or at times when re-roofing efforts are performed.

4. Vegetation Control. The high vegetative fuel load in the firestorm area also greatly contributed to the fire's rapid spreading. Strict control of vegetative fuel loads in high hazard fire areas would reduce future fire risks. Additional precautions should be taken to rid high hazard areas of fuel load caused by severe winter freezes.

5. Emergency Backup Power and Pumping. Several pumping plants lost PG&E power during the firestorm. If PG&E power had not been lost, at any time at any pumping plant, or if the water utility had immediate use of emergency backup power, the amount of additional water available to fight the fire would have been incremental: perhaps an additional 1,650 gpm split between five higher elevation areas.

The water utility had sufficient numbers of back-up generators and portable pumps to serve the affected pumping stations - however, due to the size and intensity of the fire, the times necessary to mobilize and install this equipment made them unusable for many hours into the firestorm.

The lower elevation areas with a lot of old smaller diameter pipe lost their main source of water supply when PG&E power failed at the Claremont Center. However, these same zones had substantial intra- and inter-zone ties, so that they were able to draw water from remote reservoirs and pumping plants outside the firestorm area. Shortage of water due to pumping plants out of service for these areas was not a major problem to fire fighters.

A.10.6 References

Olson, R., Eidinger, J., Goettel, K., and Foster, B., 1992. East Bay Hills Firestorm Response Assessment, Phase II Final Report, prepared for East Bay Municipal Utility District.

Standard Schedule for Grading Cities and Towns of the United States with Reference to their Fire Defenses and Physical Conditions, National Board of Fire Underwriters, 1956.

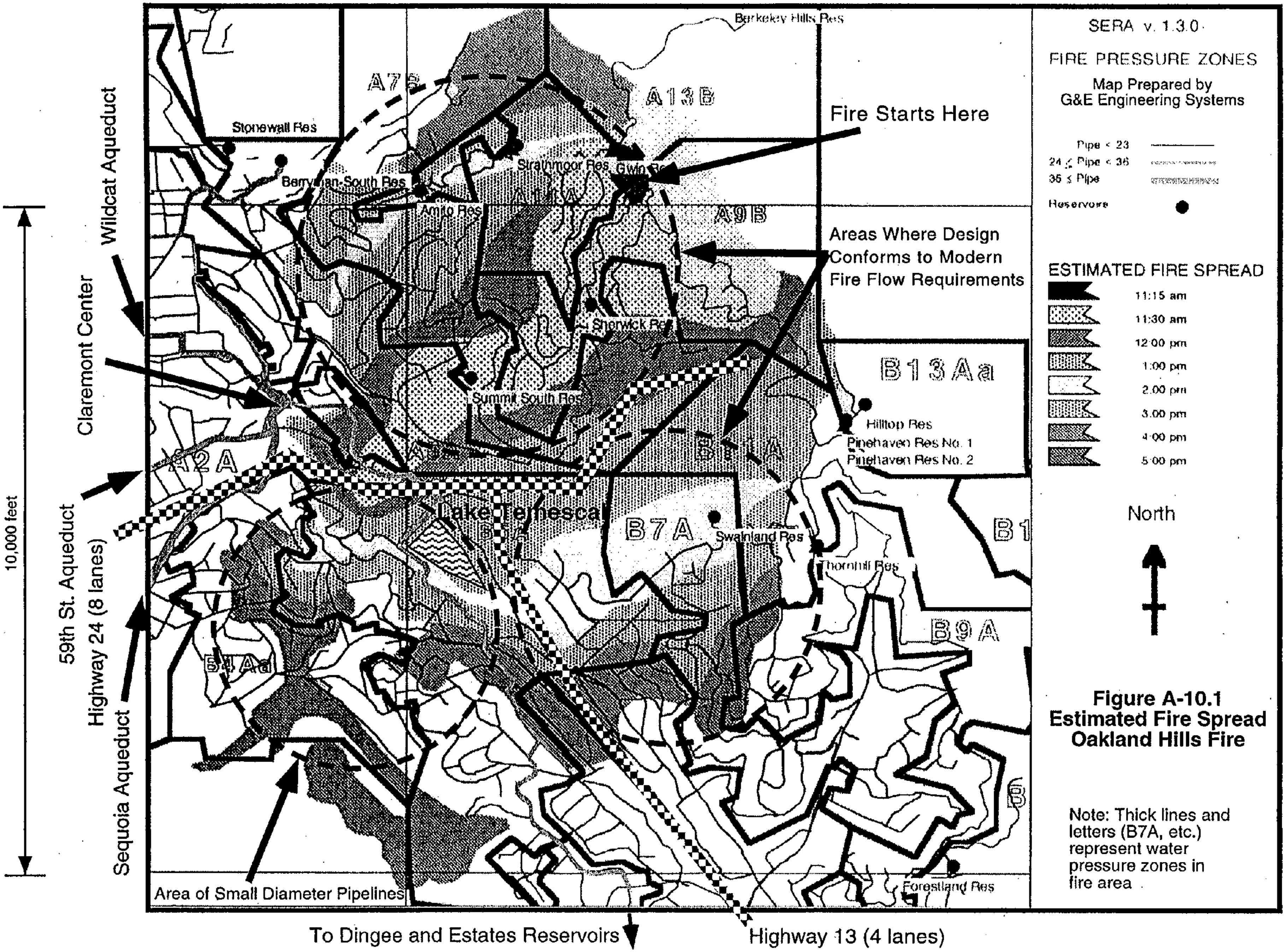

Figure A-10.1
Estimated Fire Spread
Oakland Hills Fire