AmericanLifelinesAlliance
A public-private partnership to reduce the risk to utility and transportation systems for natural hazards

Seismic Fragility Formulations For Water Systems

Part 1 – Guideline

April 2001

ASCE
American Society of Civil Engineers

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Acknowledgments

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1.0 Introduction

The American Lifelines Alliance (ALA) was formed in 1998 under a cooperative agreement between the American Society of Civil Engineers (ASCE) and the Federal Emergency Management Agency (FEMA). In 1999, ALA requested G&E Engineering Systems Inc. to prepare methods that describe the potential for damage to water transmission system components from earthquake hazards. Water transmission systems transport water from a source (e.g., well, lake, reservoir) to the delivery point—such as a storage tank—within a distribution system. The resulting damage algorithms from these methods can be incorporated into software programs to perform earthquake loss estimates.

1.1 Background

A fundamental requirement for assessing the seismic performance of a water utility is the ability to quantify the potential for component damage as a function of the level of seismic hazards. The term *vulnerability relationship* is used to refer to a general deterministic, statistical, or probabilistic relationship between the component's damage state, functionality, economic losses, etc., given some measure of the intensity of the earthquake hazard. The relationship between the probability of component damage and the level of seismic hazard is referred to as a *fragility relationship* or *fragility curve*. The relationship between economic losses associated with damage and the level of seismic hazard is normally referred to as a *loss relationship* or *loss algorithm.* The use of *vulnerability relationship* in this report is limited to relationships expressing the likelihood of experiencing a particular damage state.

Estimating damage using vulnerability relationships is improved when the relationships accurately capture conditions and characteristics of the particular system components. There is considerable project experience from implementing such refinements within industry, consulting and academic communities, although no specific procedures or guidelines exist for such refinements. A consequence of this lack of guidance is the inability to directly compare the potential earthquake damage for water transmission systems among a diverse population of system owners and users. Lack of uniformity in risk assessment impedes the prioritization of what actions should be taken to reduce damage and where resources should be focused to improve performance.

1.2 Project Objective

The goal of this project was to develop detailed procedures that can be applied to any water transmission system in order to evaluate the probability of damage from earthquake hazards to various components of the system. The products of this project include the fragility curves for each type of component and appendices containing the data used in the analyses, comparisons of the fragility curves with those prepared by other researchers in the past, examples of application of the methods, and a description of the statistical analysis methods used in developing the fragility curves.

The fragility curves presented in this report were formed in a transparent way. This means the fragility curves were documented with all raw data, which will allows revisions from new information that may become available in the future.

1.3 Project Scope

The following components of a generic water transmission system were considered in the scope of this project:

- Water conveyance systems (pipelines, tunnels and canals)
- Above-ground cylindrical storage tanks
- Portions of the conveyance control and data acquisition (SCADA) system that are located along the conveyance system
- Flow control mechanisms (e.g., valves and gates)

For each component, the likely damage states and corresponding fragility functions are presented.

The following components were excluded from the scope of this project:

- Pumping plants
- Treatment plants
- Diversion structures
- Central control facilities
- Buried or in-ground reservoirs
- Dams
- Hydroelectric plants
- Buildings
- Transportation and utility systems that support the operation of the water transmission systems (e.g., roads, bridges, outside electrical power, outside telecommunications, etc.)

Although it is beyond the scope of this report to describe how to calculate earthquake hazards, Chapter 3 presents a brief summary of the topic in order to establish the hazard parameters needed to use the fragility functions.

The fragility curves presented in this report consider both uncertainty and randomness. Uncertainty and randomness stem from both the characterization of the earthquake hazard as well as the performance of the component itself to a particular level of hazard.

Two generic examples of the product expected from the scope of work are illustrated in Figures 1-1 and 1-2. An approach commonly used for conveyance systems is to define a baseline vulnerability relationship and modify this relationship to account for the specific configuration of the system, as illustrated in Figure 1-1. In Figure 1-1, the "component" is a segment of the conveyance system with constant properties (e.g., material, size, joint type, etc.) and uniform hazard exposure. The length of the segment may vary from tens to thousands of meters.

The hypothetical form of the vulnerability function in Figure 1-1 does not provide a probability of failure for a particular segment. However, defining the error associated with estimating the damage measure allows the likelihood of the occurrence of damage to be computed. Using Figure 1-1, assuming that the damage measure is a break in a unit length of a conveyance component causing loss of conveyance, a hazard measure of 5 corresponds to a mean of 0.6 breaks/length and the mean plus one standard deviation is 1.0 breaks/length. If one knows the underlying probability distribution, the mean and standard deviation permit the probability of experiencing a specific number of breaks in the segment to be estimated.

For aboveground cylindrical storage tanks, vulnerability relationships can have a more analytical basis, since basic parameters such as tank height, diameter, wall thickness, fluid level and anchorage capacity can be used to estimate tank stresses and displacements. These response parameters can be related to the probability of experiencing a particular tank damage state (e.g., buckling, excessive uplift, roof damage) as a function of earthquake ground motion. The results can be expressed as illustrated in the hypothetical relationship plotted in Figure 1-2.

Considerable uncertainty can exist when quantifying vulnerability relationships. The procedures developed in this report provide the baseline or median vulnerability expressions as well as the process and basis for quantifying the uncertainty associated with the relationships.

1.4 Uncertainty and Randomness

The fragility formulations for water system components provide explicit consideration of uncertainty and randomness. Part of the total uncertainty and randomness stems from the underlying earthquake hazard and part stems from the specific water system component.

There are at least two ways of tracking the uncertainty and randomness in these evaluations:

Method 1: Track the dispersion parameters for both the earthquake hazard and the component. Combine these two into a total estimate of dispersion. Carry this total dispersion value through the analysis.

- This approach is convenient in that the complexity of the analysis is simplified into just a few terms (e.g., medians and betas) of a component. The HAZUS computer code [FEMA, 1999] follows this approach.
- A drawback of this approach is that it is not flexible enough to deal with distributed systems like water systems that are composed of links and nodes. The form of the dispersion for each component (either a component of a link or a component of a node) may differ. Fault tree logic used to assess whether a specific link or node is in various possible damage states could make it inappropriate to combine dispersions of individual components in a simple mathematic way.

Method 2: Track the dispersion parameters for both the earthquake hazard and the component. Evaluate each component separately using a Monte Carlo simulation technique. For each simulation, combine the results for each component into a global performance specific to a link or a node; then combine the performance of all links and nodes using a suitable system model to establish how well the overall system performs. Finally, repeat this analysis for many simulations and track the range in overall system performance.

- This approach can conveniently handle any form of dispersion model for specific components as well as track the entire system analysis for individual dispersions of individual components and localized ground hazards.
- A drawback of this approach is that it requires more computation effort than Method 1.

This report does not recommend one method over the other. Unless specifically noted, this report provides dispersion parameter information that can be used in Method 1. If the user wishes to de-aggregate the total dispersion into that only associated with the component, then the dispersion associated with the hazard must be removed. This is usually done by applying a SRSS rule, which is explained in detail in Appendix E. While this combination method is not always rigorous, it may be suitable for the application being considered by the user. Since all the raw data used to establish the fragility functions is presented in this report, the user can analyze the empirical data to establish fragility curves suited to a specific hazard. These might include high magnitude subduction zone earthquakes that usually have longer durations than California earthquakes, or eastern United States earthquakes that typically have larger uncertainties in ground motions than those associated with California earthquakes.

1.5 Outline of this Report

In order to calculate loss estimates of water systems three types of information are needed:

- Inventory information: Section 2 describes the issues involved.
- Seismic hazard information: Section 3 describes the issues involved.
- Fragility models: Sections 4 through 8 describe the fragility models.

The raw data for the fragility models is presented in Appendices A (pipes), B (tanks), C (tunnels) and D (canals).

Appendices A through D present commentary and comparisons of the fragility models to those in the literature.

Appendix E presents some basic mathematical models that are used in this report, covering liner regression and the normal and lognormal distributions.

Appendix F presents an example application of pipeline fragility models for a water transmission system exposed to ground shaking, liquefaction and landslide hazards.

Appendix G presents an alternate method to compute fragility curves using Bayesian analysis instead of standard regression methods.

1.6 Terms and Definitions

Certain terms used in the water system methodology of this report are defined below.

Conduit. A free-flowing conduit can be an open channel or ditch, or may be a tunnel flowing partially full. A pressurized conduit can be a pipeline or tunnel flowing under internal pressure. An open channel can be a canal or a flume.

Canal. A free-flowing conduit, usually open to the atmosphere, and usually at grade. A canal may be lined or unlined.

Damage Algorithm. Same as fragility curve.

Distribution Storage Reservoir. Most water systems include various types of storage reservoirs in their distribution systems. Storage reservoirs can be either tanks or open cut reservoirs. Fragilities developed in this report cover at-grade and elevated steel, concrete and redwood storage tanks.

Distribution System. The system that delivers treated water to customers for end use. Most water distribution systems in the US deliver treated water for drinking, sanitary, irrigation, commercial, industrial and fire flow purposes. In some cities, separate distribution systems are built to deliver reclaimed water for irrigation or industrial purposes and to supply water to fire hydrants. The fragility formulations in this report can be used for these additional water systems, which comprise a very small percentage of all distribution systems.

Fragility Curve. A mathematical expression that relates the probability of reaching or exceeding a particular damage state, given a particular level of earthquake hazard.

Flume. A free-flowing conduit, usually open to the atmosphere and usually elevated. A flume is typically built from wood or metal with wood or metal supports. The seismic performance of flumes is not covered in this report.

Hazard. An earthquake hazard can include ground shaking, response spectra, peak ground velocity, peak ground acceleration or permanent ground deformation.

Open Cut Reservoir. Many water systems store water in open cut reservoirs. “Open cut” simply means that the reservoir is built by creating a reservoir in the natural lie of the land, often with one side of the reservoir made up of an earthen embankment dam. Many open cut reservoirs are enclosed by adding a roof so that treated water inside is protected from contamination from outside sources. A few open cut reservoirs in treated water systems are open to the air, and water in these reservoirs usually must be treated before being delivered to customers. This report does not provide fragility formulations for this type of reservoir. Such fragilities would have to consider the performance of earthen embankment dams, roof structures and, possibly, inlet-outlet towers.

Pumping Plant. A facility that boosts water pressure in both transmission and distribution systems. The plant is usually composed of a building, one or more pumps, electrical equipment, and, in some cases, backup power systems. This report does not provide fragility curves for pumping plants or pumping plant components.

Raw Water. Water as it is found in nature. This water may be in lakes, rivers or below-ground aquifers. Raw water is generally not used for drinking water because it does not conform to water quality requirements set by various Federal and State agencies.

Tanks. A vessel that holds water. Water tanks are usually built of steel, concrete or wood—most often redwood. Tanks can be elevated by columns; built “at-grade” to rest directly on the ground or on a foundation on the ground; or buried. Also, in some smaller parts of distribution systems, water can be stored in pressure tanks, which are small horizontal pressure vessels on supports, at grade. This report provides fragility curves for most kinds of tanks.

Transmission System. A system that stores “raw” water and delivers it to water treatment plants. Such a system is made up of canals, tunnels, elevated aqueducts, buried pipelines, pumping plants and reservoirs.

Treated Water. Water that has been processed to meet water quality requirements set by various Federal and State agencies. Under normal conditions, water flowing out of taps in residences is treated water. If treated water becomes contaminated because of damage to the water system during an earthquake, water agencies issue "boil water" alerts to their customers.

Treatment System Facilities. Large centralized water treatment plants are common to most cities in the US and used when the raw water source is lakes or rivers. Small, local treatment facilities at well sites are common when the raw water source is a below-ground aquifer. In some cities, treated water is stored in open-air reservoirs and requires some secondary treatment before being delivered to customers. This report does not provide fragility curves for treatment plants.

Vulnerability Function. Same as fragility curve.

Wells. Used in many cities as both a primary and supplementary source of water, wells include a shaft from the ground surface to the aquifer, a pump to bring the water up to the surface, equipment used to treat the water, and a building to enclose the well and equipment. This report does not provide fragility curves for wells.

1.7 Abbreviations and Acronyms

Abbreviations used in this report are listed below.

AC	Asbestos Cement
ALA	American Lifelines Alliance
ANSI	American National Standards Institute
ASCE	American Society of Civil Engineers
CI	Cast Iron
COV	Coefficient of Variation
cm/s	centimeter per second
DI	Ductile Iron
EBMUD	East Bay Municipal Utility District
EPA	Environmental Protection Agency
FEMA	Federal Emergency Management Agency
fps	feet per second
G&E	G&E Engineering Systems Inc.
GIS	Geographical Information System
HDPE	High Density Polyethylene
LADWP	Los Angeles Department of Water and Power
ln	natural logarithm

M	Magnitude (moment magnitude unless otherwise noted)
mm	Millimeter
MMI	Modified Mercalli Intensity
PGA	Peak Ground Acceleration (g)
PGD	Permanent Ground Deformation (or Displacement) (inches)
PGV	Peak Ground Velocity (inches/second)
PLC	Programmable Logic Controller
PVC	Polyvinyl Chloride
RR	Repair Rate (Repairs per 1,000 feet or Repairs per kilometer. RR = λ.)
RS	Response Spectra
RTU	Remote Terminal Unit
SCADA	Supervisory Control and Data Acquisition
SRSS	Square Root of the Sum of the Squares
TCLEE	Technical Council on Lifeline Earthquake Engineering
USGS	United States Geological Survey
WTP	Water Treatment Plant

1.8 Units

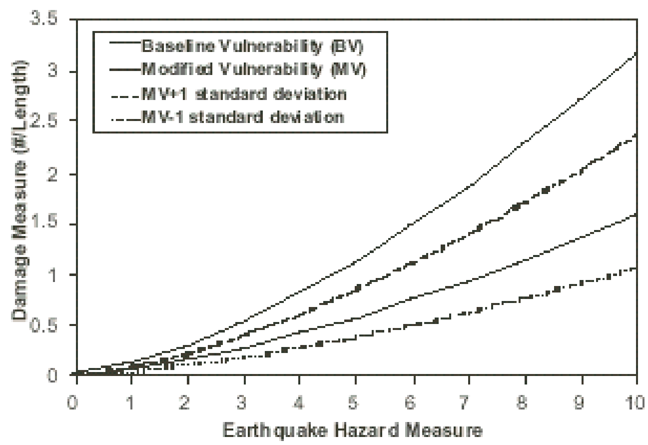
Both common English and SI units are used in this report.

Most water pipelines in the United States are sized by diameter using inches as the unit of measure. For example, distribution pipes are commonly 6 or 8 inches in diameter. As these are nominal diameters, the actual measured diameter can vary, depending on lining and coating systems, the pipe manufacturer and the material used. Conversion of a 6-inch diameter pipe to a 152.4 mm diameter pipe implies an accuracy that does not exist; conversion of a 6-inch diameter pipe to a 150 mm diameter pipe implies that the pipe was purchased in a metric system, which in most cases it was not (at least in the US). Thus, English units are used where conversion to SI units would introduce inaccuracies.

1.9 References

FEMA, 1999, HAZUS 99, *Earthquake Loss Estimation Methodology*, developed by the Federal Emergency Management Agency with the National Institute of Building Sciences.

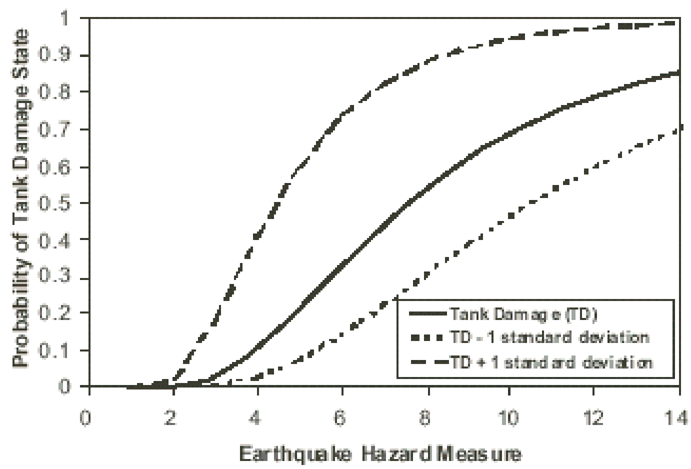
1.10 Figures



DM=f(D, t, d, J, M, S, E, XX)

DM = damage measure
D = diameter
t = wall thickness
d = depth of soil cover
J = joint type
M = material
S = site characteristics
E = earthquake hazard measure
XX= other parameters to be determined

Figure 1-1. Idealized Vulnerability Relationship for Water System Pipelines



PD = f (H, D, t, M, F, A, E, S, YY)

PD = probability of damage state occurring
H = height
D = diameter
t = wall thickness
M = material
F = fluid
A = anchorage
E = earthquake hazard measure
S = site characteristics
YY = other parameters to be determined

Figure 1-2. Idealized Vulnerability Relationship for Water System Components

2.0 Inventory

To calculate a loss estimate for a water transmission system, the analyst first collects an inventory of the components and the seismic hazards that might affect the system. This is a key step in performing the analysis. If a rough description of the inventory or hazards is collected, then only a rough estimate of how the water system will perform in an earthquake will be possible.

Depending on the objective of the loss estimation effort, the analyst may or may not have access to all the desired inventory information. For example, the material used to construct the pipelines might not be known with certainty unless original pipeline drawings are collected. Since pipeline performance is likely to be a function of the material used in construction, the analyst might assume “average” quality construction and choose a fragility curve that represents average quality materials. The uncertainty in the analysis results will increase, but this may be satisfactory if the analyst is trying to do a rough “first cut” type of evaluation.

To calculate a loss estimate for a water system, one first has to collect inventory information about the water transmission system components. The following sections describe the required input.

2.1 Study Area

The area where the loss estimation is being performed is called the study area. The study area could represent a city, a county, a group of counties or even multiple states, as appropriate.

In some water systems, key parts of the system are located some distance from the immediate area of concern. The user must consider how big to make the study in order to include all vital parts of the water system. The study area should encompass all areas with ground shaking projected to be 0.05 g or higher.

A Geographical Information System (GIS) may be a convenient way to illustrate the results of a loss estimation.

2.2 Aqueducts

Raw water is delivered to water treatment plants in large water conveyance facilities, commonly called aqueducts. An aqueduct is made up of one or more of the following elements:

- **Elevated Pipes** are large-diameter (4-foot to 7-foot) pipes supported on bents. Elevated pipes are often used in areas that traverse poor soils, and the bents are often supported on piles that extend to competent materials. An elevated pipe is usually made of riveted or welded steel pipe material. Riveted pipes were common before 1940. Above-ground welded steel pipe is made of either water-grade (poorer quality) or oil-grade (better quality) material. Pile supports can be made of wood, concrete or concrete-encased steel.
- **Buried Pipes** are large-diameter pipes buried 3 to 15 feet or deeper in the ground. Materials are often concrete pipe with steel cylinder or steel. Steel is either riveted or welded, most often using water-grade materials.

- **Canals.** Canals can be formed by cutting a ditch into the ground, building up levees, or a combination of the two. Most often, canals are concrete-lined to reduce water losses. Canals can traverse both stable and unstable geologic conditions. Unstable geologic conditions include liquefaction zones, landslide zones and fault-crossing zones.
- **Tunnels.** Tunnels can be classified as one of four types: rock tunnels, tunnels through alluvium with good quality liners, tunnels through alluvium with average quality liners, or cut-and-cover tunnels. Tunnel liners can be damaged by strong ground shaking or fault offset, and portals can be damaged from landslide. It is conceivable, although not common, for a cut-and-cover tunnel to traverse soils prone to liquefaction.
- **Flumes.** Flumes are open-channel sections that carry water in elevated structures. The channel sections are commonly made of wood or metal. The support systems can be built of wood, concrete or steel. The support structures might be a few feet high where the flume runs along a contour, or very tall where the flume crosses a creek or river. Flumes are specialized structures and are not specifically addressed in this report.

For purposes of loss estimation, the following attributes may be needed for each aqueduct:

- **Location.** End and interior points along the length of the aqueduct within the study area are needed to describe location. If the aqueduct crosses through geologically unstable areas (e.g., liquefaction zones, landslide zones), then specific x-y pairs are needed at the start and end of that area.
- **Type.** The aqueduct should be described as being elevated, buried, a canal or a tunnel. If the aqueduct is elevated or buried, the pipe materials should be established. If the aqueduct is a canal, it should be determined whether the canal is open cut and concrete-lined, open cut and compacted earth-lined, or built up using levees. If the aqueduct is a tunnel, it should be determined to be lined or unlined and the type of liner should be established.
- **Multiple Aqueducts.** If the aqueduct is composed of multiple lines, each parallel pipeline, canal or tunnel should be considered. For example, a 7.5-degree USGS topographical map may indicate a single line for an aqueduct, but a more detailed water agency map may show multiple parallel pipelines.
- **Appurtenances along the length of the aqueduct**. This includes various turnouts, gates, valves, etc. Often ignored for a simplified earthquake loss estimate, these may be important if there are particular component vulnerabilities, or if a system model that includes connectivity is to be used.
- **Gravity systems or pumped systems.** Gravity system aqueducts deliver the flow from higher elevations to lower elevations, and do not need any pumping to move the water. Pumped-system aqueducts require pumps along the length of the aqueduct to keep the water moving. Some gravity aqueducts may include pumps along their length, where the pumps are occasionally used to increase flow, but are not required for minimum flow rate operations. If an aqueduct requires pumping and the pumping plant is located in the study area, then the pumping plant should be located and evaluated. Seismic evaluations of pumping stations are not addressed in this report.

2.3 Distribution Pipelines

Distribution pipe refers to buried pipe that carries water to customers and fire hydrants. For a detailed loss estimation study, the user digitizes the actual locations of all such pipe along with its attributes.

The following information is optimally needed for the seismic evaluation of distribution pipe. Many different pipe materials are currently in use in water systems throughout the US. Based upon review of water systems serving the Seattle, Portland, San Diego, Los Angeles and the San Francisco Bay areas, for example, a single set of inference rules that will not be valid for any single water agency. The water agency serving the city of San Francisco uses cast iron and ductile iron pipe; the water agency serving the cities on the east side of San Francisco Bay uses welded steel, cast iron, plastic and asbestos cement pipe. The trend of using different pipe materials is also evident in the greater Seattle area—one agency uses ductile iron pipe while another uses asbestos cement pipe.

Despite these differences, some assumptions about pipe materials can be made.

- Probably about 75% to 90 of all pipe in the US installed prior to 1945 is cast iron.
- Other older vintage pipe materials include riveted steel, wood, and wrought iron. If actual pipe material information is unavailable, it is reasonable to assume that all neighborhoods developed before 1945 have cast iron pipe.
- The most common joinery methods for cast iron pipe is the use of “bell and spigot” connections. These types of connections are also called “segmented” construction. These joints are made leak-tight using either cement, lead or rubber gasket materials. Cemented joints are common and can be used as a default.
- For pipe installed since 1945, a variety of materials have been commonly used throughout the US.
- Asbestos Cement (AC) pipe was often used from about 1945 to 1985 for pipe diameters up to 12 inches. AC pipe is no longer used for new construction. Two types of joints are common with AC pipe: rubber gasket, which is more common, and cement, which is less common. AC pipe is segmented pipe.
- Polyvinyl chloride (PVC) pipe for diameters up to 12" is gaining wider use at many water agencies, particularly for installations made since 1985.
- Welded steel pipe has been in use since the early 1900s, particularly for larger diameter pipe (12" diameter and larger). Welds made prior to the 1940s using the oxyacetylene welding technique were often made with poor quality control and therefore exhibit severe welding defects compared to modern practice; good quality oxyacetylene welds can be as good as early arc welds. The quality of the welds can be ascertained through inspection and play an important role in establishing the seismic ruggedness of welded steel water distribution pipe.
- Ductile iron pipe has been in use since the 1940s for all pipe diameters. Ductile iron pipe can have either segmented or mechanically restrained joints.

- Concrete cylinder pipe has been in use since the 1920s for larger diameter pipe (36" diameter and larger). Concrete cylinder pipe uses segmented joints, but some installations incorporate a thin steel plate interior to the concrete welded at the joints.
- Some water agencies continued to use cast iron pipe through the early 1970s.
- Other pipe materials in use include riveted steel pipe, wrought iron pipe and copper pipe, particularly for customer-side pipe from the meter to the structure.

The diameter of distribution pipe is important both in terms of pipe damage algorithms and post-earthquake performance of the entire water system. The nominal pipe diameters used for distribution pipe in the US are:

- Local distribution systems use some 4" and a lot of 6" and 8" pipe. Local distribution pipes are those that most often provide connections to structures and fire hydrants. Generally speaking, if a small-diameter distribution pipe breaks, only the customers directly connected to that pipe will be out of service once the broken pipe is valved out of the system.
- Backbone pipes in distribution systems use 12", 16", 20", 24", 30", 36", 42", 48", 54" and 60" diameter pipes. Backbone pipes are those that connect pressure zones from treatment plants to pumping plants to storage reservoirs. Generally speaking, if backbone pipes break, many customers will be out of service.

Other pipe attributes that may be developed when collecting inventory data include: leak history, encasement, corrosion protection systems, location of air valve and blow offs, etc. These attributes may yield some extra information as to the pipeline's fragility, but they may not be available to the analyst in all cases.

2.4 Storage Tanks

Storage tanks can be located at the start, along the length or at the end of a water transmission system. Their function may be to hold water for operational storage, provide surge relief volumes, provide detention times for disinfection, and other uses.

The following information will be needed to evaluate the storage tanks:

- Seismic hazards at the tank site. This includes the type of soil (e.g., rock, firm soil, soft soil) and the susceptibility of the site for landslide and liquefaction.
- Construction. A field survey should be done to assess the tank’s configuration, including the style of foundation, the presence of side-located inlet-outlet pipes (and any flexible couplings these may have); the style of roof system; the style of tank anchorage, if any; and estimated volume (height and diameter). The survey requires a drawing review to affirm the structural properties of the tank, such as actual anchorage details, especially for concrete tanks. Other properties include hoop prestressing, wall thicknesses and various structural details of the roof system. The operating function of the tank should be reviewed to ascertain whether the tank is normally kept full or nearly full, as is most common, or kept less than full, as with surge or other tanks.

Several types of water tanks are in use today in the United States.

Steel Tanks. These tanks, when at grade, can range in size from very small (under 200,000 gallons) to quite large (14 million gallons or larger). Elevated steel tanks are limited in capacity to about 2 million gallons, although some elevated tanks hold up to 5 million gallons. At-grade steel tanks can be either anchored or unanchored. Elevated steel tanks typically have lateral load resistant capacity for wind or earthquakes.

The walls of steel tanks are built from sheet steel in courses. A course is a level of the tank, often 8 to 10 feet tall. The number of steel sheets that comprise a course will vary based upon the outside circumference of the tank, and the length of each sheet of steel. The more common method to join these sheets of steel is to weld them together. On smaller volume tank (mostly under 200,000 gallons), it is not uncommon to use bolts to join the sheets; in a few older cases, rivets may be used.

The roofs of steel tanks are either made of steel or wood. Wooden roofs are more susceptible to damage in earthquakes than steel roofs. It is possible, although uncommon, to have steel tanks without roofs.

Concrete Tanks. Concrete tanks can be either at-grade or buried. Some older concrete tanks are reinforced concrete and many are post-tensioned. Until the 1980s, few at-grade post-tensioned concrete tanks were designed for significant seismic forces; the joint detail at the bottom of the walls specifically requires the walls to be able to slide relative to the foundation to accommodate the post-tensioning process.

Wood Tanks. Wood tanks are generally at-grade and are limited in capacity to about 400,000 gallons. Smaller capacities can be used in elevated tanks. These are commonly used in California but are uncommon in other parts of the nation. Most wood tanks in California are made from redwood, but the actual type of lumber used probably has little effect on seismic capacity. Wood tanks are less expensive to construct than either steel or concrete tanks and are generally unanchored.

Open Cut Reservoirs. An open cut reservoir is made by cutting into the ground, and typically, an earthen embankment dam completes the reservoir. These reservoirs range in storage capacity from a few million gallons to well over 100 million gallons. They may or may not include roof structures. The roofs of many treated water reservoirs were installed in the 1960s and 1970s to meet EPA water quality regulations. They are often lightweight and supported on precast columns at regular spacing. They often have large vents, resulting in a “stepped” roof design, and therefore do not have a diaphragm to distribute seismic loads to the end walls.

2.5 Tunnels

Both raw water and treated water distribution systems may use tunnels. Tunnels may be particularly prone to earthquake damage if they cross faults, or if their portals are in landslide zones. To a lesser extent, some damage to tunnel liners can occur from strong ground shaking.

For purposes of developing fragility curves, tunnels are classified into one of two categories: bored tunnels and cut-and-cover tunnels. Bored tunnels include those with various types of liner systems or without liners. More modern tunnels may have been constructed by tunnel boring machines, while older tunnels were constructed by a variety of methods. Section 6 deals with bored tunnels. Subclassifications of bored tunnels are made based on liner system and geologic conditions.

2.6 Canals

Canals are sometimes used as components of a larger water transmission system. For example, the California River Aqueduct brings water from the Colorado River to Los Angeles and is composed of the following main line components: 92 miles of tunnel, 55 miles of cut-and-cover conduit, 62.4 miles of lined canal, 29.7 miles of pressure conduits (29.7 miles) and 1 mile of unlined canal.

The basic nomenclature and design features of canals is adopted from McKiernan [1993]. The possible impact of canal design features on earthquake performance is noted.

It is useful to summarize why canals are sometimes used instead of pipelines. Canals are operated at atmospheric pressure, and tend to be larger than pipelines operated under pressure. The advantages of using a canal include the possibility of construction with locally available materials, longer life than metal pipelines, and lower loss of hydraulic capacity with age. The disadvantages include the need to provide the ultimate flow capacity initially and the likelihood of interference with local drainage.

Artificial channels for the conveyance of fluids fall into two categories: free-flow or pressure conduits. Free-flow conduits guide the fluid as it flows down a sloping surface, while pressure conduits confine and guide fluid movement under pressure. Free-flow conduits may be simple open channels or ditches, or pipes or tunnels flowing partially full. Pressure conduits such as pipelines are covered in Section 4. Tunnels can be free-flow or pressurized, and are covered in Section 5.

The cross-sectional shape of a free-flow conduit or canal is usually governed by a combination of cost and hydraulic capacity factors. A square conduit is hydraulically inefficient, and its flat sides are structurally undesirable because of the excessive use of materials for a given strength. A semi-circular cross section, open at the top and flowing full, is the most hydraulically efficient section, but this shape is rarely used because of construction condition. Given these issues, the most common shape of a canal has traditionally been trapezoidal.

Cost is almost always a factor in the initial design of canals. All other factors being equal, the smaller the cross-sectional area of a canal, the lower the cost. This means that designers will try to maximize the velocity of the water going through the canal. The maximum safe velocities for concrete-lined open channels carrying clear water can exceed 40 feet per second (fps), while safe velocities of 10 to 12 fps have been used in design. Thin metal flumes may be damaged by coarse sand or gravel at 6 to 8 fps. If the water carries an appreciable amount of silt in suspension, too low a velocity will cause the canal to fill up until the capacity is impaired. If canals are unlined, then excessively high velocities can scour of the canal, which should be avoided.

Water loss from leakage, absorption and evaporation is an important factor in canal design. Leakage from well-constructed and well maintained concrete, wood and metal conduits is relatively small; however, no conduit is completely tight. In long-lined systems, the accumulation of even small leakage may be important. Target leak rate allowances for conduits of 300 to 400 gallons per inch diameter/per mile/per day are not unheard of under normal operating conditions. For example, for a 120" diameter conduit, the target leak rate is 120 x 300, or 36,000 gallons leakage per mile/per day.

Earth canals have traditionally been trapezoidal in form, but with modern materials and construction facilities, curved bottoms are possible. Side slopes are determined by the stability of the bank materials and are often based on experience. The heights and widths of banks are determined by freeboard and stability requirements. Typical unlined trapezoidal canal sections are shown in Figure 2-1. Typical design factors for canals are as follows:

- The side slopes of cuts and fills not exposed to the action of water must conform to the angle of repose of the materials, with allowance for possible saturation by seepage. The steepest safe slopes are most economical for initial design. If earthquake-induced loading has not been factored, especially under saturated condition, then failure of these side slopes is a credible failure mode. Failure of side slopes could lead to loss of an adequate amount of freeboard, reduction in flow in a cross-sectional area, increase in sediment transport, and other concerns.

- An adequate amount of freeboard must be provided to accommodate the accumulation of sand or silt, growth of moss or other vegetation, centrifugal forces on curves, wave action, increase in flows at diversions, inflow of storm waters, etc. Slumping of freeboard is credible under earthquake loads, and if adequate freeboard does not remain after the earthquake, the canal may need to be operated at lower flow rates or shut down for repairs. The lower limit for freeboard is typically 1 foot for small canals to as much as 4 feet for large canals. For lined canals, the top of the lining is not usually extended to the full height of the bank freeboard.

- The width of the bank must be wide enough to provide the embankment with sufficient strength to resist internal water pressure and to prevent the free escape of water by seepage. The top width is usually about equal to the depth of the water, with a minimum of 4 feet or, if a road is required, 12 feet. Embankments exposed to considerable water pressure are wider and should be compacted.

- Deep cuts may yield more materials than are needed for the banks. If the excess materials or spoils are left next to the canal, a level space, or berm, is typically provided to protect the waterway from sloughing materials from the spoils. If not properly designed, spoil banks could slump under earthquake loading, sending materials into the canal.

- A canal may be lined. A liners prevents excessive water loss by seepage; eliminates having to pipe water through or under banks; stabilizes the banks; prevents erosion; promotes continued movement of sediments; facilitates cleaning; helps control the growth of weeds and aquatic organisms; reduces flow resistance; avoids waterlogging of adjacent lands; and promotes economy by reducing the need for excavation.

- The Bureau of Reclamation [Bureau of Reclamation] established design guidelines for various types of canal liners. Four types are generally in use: unreinforced concrete, asphaltic concrete, reinforced concrete and gunite. Typical thicknesses are from 1.25

inches for very low flow rates to 4.5 inches for high flow rates. Reinforcement is rarely used for usual irrigation canals unless needed for structural reasons. Temperature stresses in concrete or mortar linings can cause buckling of the liner, but this is usually not important; thus, expansion joints are not included except at junctions to rigid structures. Except in heavily reinforced liners, cracking from normal loading cannot be avoided. Lightly reinforced liners can be used to control cracking. Because of costs, even light reinforcement is often omitted, and cracking is controlled placing a weakened-pane-type joint or “sidewalk” groove formed in the concrete to a depth of about one-third of the lining thickness. High levels of ground shaking or any form of PGD could lead to excessive cracking of a liner.

The potential for damage from a heavily cracked liner depends on the original purpose of the liner. If the only function of the liner was to control the growth of weeds, such cracks may be acceptable for an extended length of time and the damage might be acceptable, If the function of the liner was to avoid waterlogging sensitive adjacent lands, the damage might not be acceptable.

2.7 Valves and SCADA System Components

Valves on major transmission pipelines are usually spaced at wide intervals. Intervals between 2,500 and 20,000 feet are not uncommon. The location of the valves is often important when deciding how a pipeline system performs as a whole; damage to a pipeline between two valves will need to be isolated by closing the valves. Once these valves are closed, customers using this pipeline will lose all water service unless an alternate water supply is available.

Obtaining the location of the valves is also important because certain pipeline mitigation strategies may involve the addition of valve actuators. Actuators include motorized or hydraulic actuators. It might also be worthwhile to determine whether the valves are located in the ground, are in the ground in reinforced concrete boxes called valve pits, or are above ground.

Historically, in-line valves have not proven to be particularly vulnerable to earthquake damage, unless the pipeline to which they are connected is also damaged. This issue is further discussed in Section 8.

SCADA system components in water transmission systems that are of interest in this project are as follows.

- **Instruments attached to the pipeline.** These may include flow and pressure devices that are sometimes installed in a venturi section of pipeline. These devices are considered to be rugged in relation to earthquake motions. However, air bubbles that could be introduced into the pipeline system may cause these instruments to give false readings.

- **Instruments attached to a canal.** These may include various types of float instruments, which are used to assess the water level in the canal. Water sloshing can affect or damage these devices.

- **Remote Terminal Units (RTUs) and Programmable Logic Controllers (PLCs).** RTUs and PLCs are most commonly solid state devices. An RTU device picks up the analog signals from one or more channels of SCADA system devices at one location. The

RTU converts these signals into a suitable format for transmission to a central SCADA computer, often at a location remote from the devices. A PLC can control when pumps are turned on or off, based on real time data or pre-programmed logic. For most seismic loss estimates, RTUs and PLCs are considered rugged and are not specifically included in the analysis.

- **Manual Recorders.** Most water systems have used manual recorders to track pressures, flows and gradient information. These recorders are still in use in many water systems today. The recorders sometimes report on the same information as the automated SCADA system, often using the same instruments. However, manual recorders rely on commercial power and will not work if commercial power is lost after an earthquake. Also, since the installation of automated SCADA system hardware is often relegated to a few locations in the water system, the manual recorder may be the only recording device at a location. This report does not include fragility information for manual recording devices.
- **SCADA Cabinet and Power Supply.** The SCADA cabinet is a metal enclosure that is mounted to a floor or bolted to a wall. If the cabinet is mounted to a floor, then floor anchorage information should be collected. If the cabinet is mounted to a wall, then the wall should be assessed as either an unreinforced masonry wall or a full structural wall. The SCADA cabinet should be inspected inside to see if all equipment is well-anchored. Most SCADA systems include battery backups. The location of the battery should be verified in the field and the installation of the battery should be noted. Some SCADA systems use Uninterruptible Power Systems (UPS) systems, which allow no loss of power to the SCADA system component if offsite commercial power is lost. The anchorage of the UPS also should be determined.
- **Communication Links.** The remote SCADA system is connected in some manner to the central location SCADA computer system. The most common links are radio, leased landlines and, to a lesser extent, microwaves; the use of public switched landlines is rare. The number and type of links should be inventoried for each SCADA system site to help assess the likelihood that the SCADA system will be able to send signals to the central location computer after the earthquake.

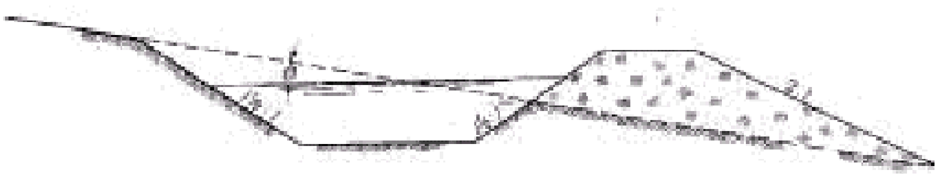
2.8 References

Bureau of Reclamation, “Linings for Irrigation Canals,” 1963.

2.9 Figures

Typical Canal Section

Typical Canal Section



Typical Canal Section

Typical Deep Cut Canal Section

Figure 2-1. Typical Canal Cross Sections

3.0 Earthquake Hazards

3.1 Background

Chapter 3 outlines the basic descriptions of geotechnical hazards that are assumed to be available or can be made available to the seismic loss estimation effort.

The state-of-the-practice in the estimation of geotechnical hazards is likely to improve over time. The current effort concentrates on the estimation of fragility to pipelines, tanks, canals, tunnels and in-line SCADA equipment.

If alternate methods are used to establish geotechnical hazards, then the fragility models will likely need to be changed. For example, as of 2001, the state-of-the-practice does not include the ability to forecast ground strains from permanent ground deformations due to landslide, lateral spreads, settlements, etc. At best, current practice can forecast regional areas with potential vertical and lateral movements.

The analyst is responsible for establishing the actual geotechnical hazards for the project at hand.

3.2 Choosing the Earthquake Hazard

Two generally accepted methods can be used for evaluating the seismic performance of an existing facility: *scenario earthquakes* and *probabilistic earthquakes*.

A *scenario earthquake* is defined as the occurrence of a particular magnitude earthquake at a particular location. The selection of scenario earthquakes usually includes large magnitude "maximum" or "maximum credible" earthquakes as well as smaller magnitude but more "probable" earthquakes. Scenario earthquakes are often considered in risk evaluations when the utility owner wishes to determine system-wide performance given a particular earthquake. Scenario earthquakes are useful for assessing the likely or maximum losses given that a particular earthquake occurs, for evaluating emergency response plans, and in meeting pre-set performance goals. By establishing the frequency of occurrence for each scenario earthquake and selecting a suite of scenario earthquakes, a loss estimate can be established on an annual basis or other suitable timeline.

A *probabilistic earthquake* is defined as the likely level of ground hazard, usually the peak ground acceleration, at a particular location within a given time frame. A common way of expressing a probabilistic earthquake is by using a hazard curve as shown in Figure 3-1. As water systems are often located over a large geographic area with varying soil types, the hazard level at different locations can vary considerably because of regional variations in soil conditions and differences in distance to the causative faults. Probabilistic earthquakes are useful for assessing expected annualized losses, establishing insurance premiums and conducting benefit cost analyses, but are not directly applicable to system-wide loss estimates.

3.3 Ground Shaking Hazard

Given that an earthquake occurs in or near a water system, some level of ground shaking hazard will occur. Ground shaking levels at locations near the fault are typically higher than ground shaking at locations far from the fault, but uncertainty in ground motions and local soil conditions can sometimes negate this trend.

Ground shaking is characterized by peak ground acceleration (PGA), peak ground velocity (PGV), or response spectra (RS) at the site location of the component. Unless specified otherwise in this report, the PGA, PGV or RS value is assumed to be for the horizontal component of motion. PGA and RS are used for above ground components and PGV is used for below-ground pipelines. Once the source location of the earthquake in known, PGA, RS and PGV can be calculated using attenuation models.

Attenuation models have been developed to account for various types of earthquakes (e.g., subduction, strike slip), types of shaking (e.g., acceleration, velocity, response spectral values for varying levels of damping), type of soil (e.g., rock, firm, soft) and other special factors such as near-field directivity effects, vertical motions or upthrust locations. Each attenuation model used should define the average level of shaking and provide a measure of the average dispersion.

This report makes no attempt to list or reference all the types of attenuation models available. Sadigh and others have defined several types [Sadigh et al, 1997].

The dispersion parameter is very important in that it plays a significant role in estimating upper and lower bounds of potential response of various water system components. Generally speaking, this parameter can be called β_R, which is the lognormal standard deviation of the ground shaking parameter. Subscript $_R$ denotes that the dispersion parameter reflects randomness.

For the evaluation of at-grade and above-ground water storage tanks, estimating the response spectral shape at the site will usually be required. Except where specifically stated in this report, the assumption is that the site-specific response spectrum is provided at 5% damping and represents the smoothened median spectral shape associated with the median peak ground acceleration for the site.

Different attenuation relationships should be used for soft soil sites, subduction zone earthquakes and earthquakes affecting the eastern US. An average of multiple attenuation relationships may also be used.

Any attenuation model used for loss estimation is assumed to provide the following minimum information:

- The median level of ground shaking expected at a specific component location, given that a particular fault breaks at a specific magnitude.
- An estimate of the dispersion in the median level of ground shaking hazard. The most common formulation in use assumes the shape of the dispersion is lognormal.

3.4 Liquefaction and Lateral Spread Hazard

Liquefaction is a phenomenon that occurs in loose, saturated, granular soils when subjected to long duration, strong ground shaking. Silts and sands tend to compact and settle under such conditions. If these soils are saturated as they compact and settle, they displace pore water, which is forced upwards. Increased pore water pressure causes two effects. First, it quickly creates a condition in which the bearing pressure of the soils is temporarily reduced. Second, if the generated pressures become large enough, material can be ejected from the ground to form characteristic sand boils on the surface. This displaced material, in turn, causes further settlement of the site.

Lateral spreading is a phenomenon that can accompany liquefaction. On many sites, the layers of liquefiable materials are located some distance below the ground surface. If the site has a significant slope, or is adjacent to an open cut such as a depressed stream or road bed, liquefaction can cause the surficial soils to flow downslope or towards the cut. Lateral spreading can be highly disruptive of buried structures and pipelines, as well as structures supported on the site.

The ideal way to evaluate the liquefaction hazard along a specific pipeline or canal right of way is to perform site-specific liquefaction analyses. For some areas of the country, liquefaction susceptibility maps have already been prepared; see Power and Holzer [1996] for a detailed bibliography of available liquefaction maps. Recent “seismic hazard zone” maps prepared by the CDMG for purposes of establishing liquefaction special study zones are generally unsuitable for loss estimation. CDMG liquefaction and landslide zones are not defined by the level of hazard, and do not verify that any hazard in fact exists [ref. http://www.consrv.ca.gov/dmg/]. Although these maps could be used as a starting point in a loss estimation effort, they should not be used with the geotechnical models presented in this report.

The simplified approach is appropriate for initial evaluations and may be sufficiently valid for a regional evaluation, but may not be suitable for site-specific evaluations.

The liquefaction analysis should provide an estimate of the probability that a specific site will liquefy and, if it does, the amount of permanent ground deformation (PGD) expected at the site. PGD can be either vertical (settlement) or lateral (lateral spread) or a combination of the two. For a combination, the vector sum value of PGD should be used with pipeline fragility curves.

For practical purposes, most regularly designed buried pipelines will sustain damage at lateral PGDs over a foot, so extreme accuracy in the lateral spread PGD parameter is not essential.

Methods used to estimate the effects of liquefaction are provided in the 1997 liquefaction workshop [Youd and Idriss, 1997].

3.5 Landslide Hazard

Landslide hazards encompass several distinct types: deep-seated and rotational landslides, debris flows and avalanche/rock falls. These types of landslides can affect water system components in different ways:

- **Buried pipelines, valves and vaults.** Deep-seated rotational and translational landslides pose a significant damage threat to buried pipelines, valves and vaults. Most previous efforts in estimating landslide-induced damage to water pipelines has been for deep-seated landslides. Debris flows and avalanches are usually not credible threats to buried structures.

- **Water storage tanks.** Deep-seated rotational and translational landslides pose a significant damage threat to at-grade storage tanks. Even a few inches of landslide-induced settlement can distort a tank enough to cause it to fail, particularly in the case of concrete tanks. Debris flows can also damage tanks if the flow is large enough and hits the tank at a high enough velocity. Avalanches and rock falls could, in some circumstances, impact sufficiently on above-ground structures to cause damage.
- **Canals.** Debris flows can be significant threats to canals and can be activated by heavy rainfalls and/or earthquakes, particularly when the ground is saturated.
- **Tunnels.** Landslides pose a serious threat to tunnels at the tunnel portal locations.

Section 3.5 discusses hazard models for deep-seated landslide movements.

This document does not present models for debris flows, rock falls or avalanches. If a particular water system component appears vulnerable to these types of landslides, then a site-specific hazard model should be developed.

The three basic steps in evaluating the deep seated landslide hazard are:

1. Develop a landslide susceptibility map.
2. Estimate the chance of a landslide, given an earthquake.
3. Given that a landslide occurs, estimate the amount and range of movement.

Landslide maps should be created by geologists familiar with the geology of the area. The methods for developing these maps range from aerial photo interpretation to field investigation to borehole evaluations. The cost can be substantial, however, especially if no other maps are available.

For some areas, landslide susceptibility maps have already been prepared. For example, the USGS has issued a number of such maps [Nielson]. Recent "seismic hazard zone" maps prepared by the CDMG for purposes of establishing landslide special study zones are generally unsuitable for loss estimation. Site specific surveys and aerial photographs can be used for specific pipeline alignments.

Earthquake-induced landsliding of a hillside slope occurs when the static plus inertial forces within the slide mass cause the factor of safety to temporarily drop below 1.0. The value of the peak ground acceleration within the slide mass required to cause the drop is denoted as the critical or yield acceleration, a_c. This value of acceleration is determined by pseudo-static slope stability analyses and/or empirically based on observations of slope behavior during past earthquakes.

Deformations can be calculated using the approach originally developed by Newmark [1965]. The sliding mass is assumed to be a rigid block. Downslope deformations occur during the time periods when the induced PGA within the slide mass, a_{is}, exceeds the critical acceleration, a_c. In general, the smaller the ratio below 1.0 of a_c to a_{is}, the greater the number and duration of times when downslope movement occurs; thus, the greater the total amount of downslope movement. The amount of downslope movement also depends on the duration or the number of cycles of ground shaking. Since duration and the number of cycles increase with earthquake magnitude, deformation tends to increase with increasing magnitude for given values of a_c to a_{is}.

The landslide evaluation requires characterization of the landslide susceptibility of the soil / geologic conditions of a region or subregion. Susceptibility is characterized by the geologic group, slope angle and critical acceleration. The acceleration required to initiate slope movement is a complex function of slope geology, steepness, groundwater conditions, type of landsliding and history of previous slope performance. At present, a generally accepted relationship or simplified methodology for estimating a_c has not been developed, but a possible relationship proposed by Wilson and Keefer [1985] could be used. Because of the conservative nature of such a model, an adjustment should be made to estimate the percentage of a landslide susceptibility category that is expected to be susceptible to a landslide. Wieczorek and others [1985] suggest such relationships. Thus, at any given location, landsliding either occurs or does not occur within a susceptible deposit, depending on whether the peak induced PGA, a_{is}, exceeds the critical acceleration, a_c.

For locations that do slide, the amount of PGD needs to be estimated.

The uncertainty in any estimated landslide PGD is governed by the uncertainty in the local induced ground acceleration. For other factors in the model, this could be roughly accounted for by increasing the ground motion uncertainty parameter to 0.5 or so, or by having a competent geotechnical engineer provide a site-specific description of the uncertainties involved. This document does not assess this uncertainty other than to note that this value may be important in terms of the overall water system loss estimate.

3.6 Fault Offset Hazard

The amount of surface displacement due to surface fault rupture can be estimated using models such as those provided by Wells and Coppersmith [1994].

Most such models predict the maximum displacement anywhere along the length of the surface fault rupture. Fault offset will vary along the length of the surface rupture from zero inches to the maximum amplitude. Given this variation, it is recommended that the maximum displacement from such models be varied along the length of the fault, from zero to the maximum, with an expected value of some percentage of the maximum displacement.

Most fault offset models provide a median estimate of the maximum displacement along the length of the fault for a given magnitude earthquake. A dispersion estimate of the amount of fault offset is usually provided with the model.

3.7 References

Newmark, N. M., “Effects of Earthquakes on Dams and Embankments,” *Geotechnique*, vol. 15, no. 2, pp. 139-160, 1965.

Nielsen, Tor H., Preliminary Photo Interpretation Map of Landslide and Other Surficial Deposits of the Richmond 7-1/2' Quadrangle, Contra Costa and Alameda Counties, California, 1975.

Power, M. S., Holzer, T. L., Liquefaction Maps, *ATC Tech Brief* No. 1, Applied Technology Council, Redwood City, CA, 1996.

Sadigh, K., Chang, C.Y., Egan, J.A., Makdisi, F., Youngs, R.R, “Attenuation Relationships for Shallow Crustal Earthquakes Based on California Strong Motion Data”, *Seismological Research Letters*, v. 68, no. 1, p. 180, 1997.

Wells, D. L., and Coppersmith, K. J., “Updated empirical relationships among magnitude, rupture length, rupture area and surface displacement,” *Bulletin of the Seismological Society of America*, 1994.

Wieczorek, G. F., Wilson, R. C., and Harp E. L., Map of slope stability during earthquakes in San Mateo County, California, U.S. Geological Survey miscellaneous investigations, Map I-1257-E, scale 1:62500, 1985.

Wilson, R. C. and Keefer, D. K., “Predicting area limits of earthquake-induced landsliding, evaluating earthquake hazards in the Los Angeles region,” U.S. Geological Survey professional paper, Ziony, J. I., Editor, p. 317-493, 1985.

Youd, T. L., and Idriss, Ed., Liquefaction workshop, NCEER, 1997.

3.8 Figures

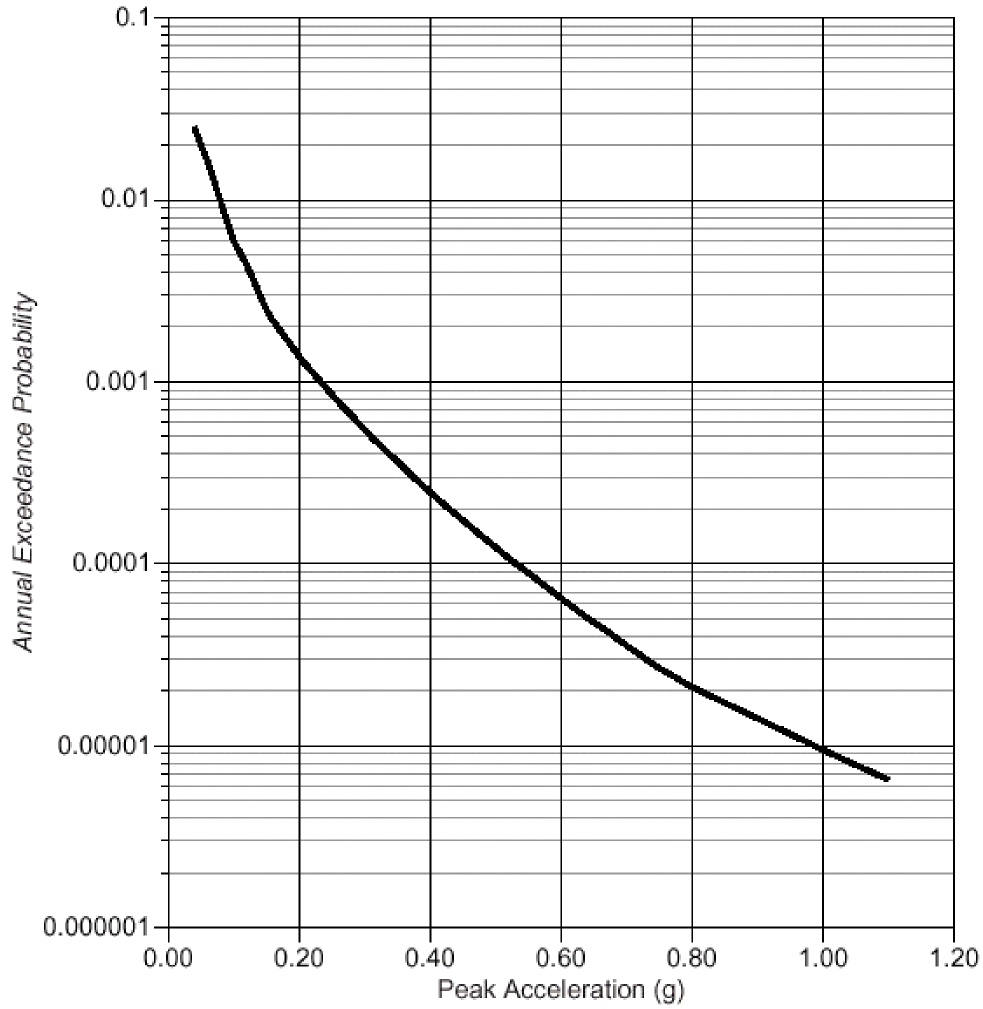


Figure 3-1. Seismic Hazard Curve

4.0 Buried Pipeline Fragility Formulations

4.1 Factors that Cause Damage to Buried Pipes

The following subsections describe the factors that lead to damage to buried pipe in earthquakes.

4.1.1 Ground Shaking

Ground shaking refers to transient soil deformations caused by seismic wave propagation. Ground shaking affects a wide area and can produce well-dispersed damage. The level of ground shaking at a pipeline location can be measured in terms of peak horizontal ground velocity (PGV).

4.1.2 Landslides

Landslides are permanent deformations of soil mass, producing localized, severe damage to buried pipe. More landslides will occur if the earthquake occurs during the rainy winter season. While some landslides may be small and displace only a few inches of soil, others may involve 100,000 cubic yards or more of soil over many feet of distance, damaging entire areas of pipelines. The amount of landslide movement is measured in terms of permanent ground displacement (PGD).

4.1.3 Liquefaction

Liquefaction is a phenomenon that occurs in loose, saturated, granular soils when subjected to long duration, strong ground shaking. Silts and sands tend to compact and settle under such conditions. If these soils are saturated as they compact and settle, they displace pore water, which is forced upwards. Increased pore water pressure causes two effects. First, it quickly creates a condition in which the bearing pressure of the soils is temporarily reduced. Second, if the generated pressures become large enough, material can be ejected from the ground to form characteristic sand boils on the surface. This displaced material, in turn, causes further settlement of the site.

Lateral spreading is a phenomenon that can accompany liquefaction-induced settlements. On many sites, the layers of liquefiable materials are located some distance below the ground surface. If the site has a significant slope, or is adjacent to an open cut such as a depressed stream or road bed, liquefaction can cause the surficial soils to flow downslope or towards the cut. Lateral spreading can be highly disruptive of buried pipelines.

Seismic soil liquefaction has the potential to occur in certain soil units and can result in permanent ground deformations. Heavy concentrations of breaks will occur in areas of liquefaction-induced lateral spreading. The orientation of the pipe relative to the ground movement can affect the amount of damage [O'Rourke and Nordberg].

The amount of liquefaction movement is measured in terms of permanent ground displacement (PGD).

4.1.4 Settlement

Pipe breaks occur due to relative vertical (differential) settlements at transition zones from fill to better soil, and in areas of young alluvial soils prone to localized liquefaction. Breaks can also occur where pipes enter tanks or buildings. The amount of settlement movement is measured in terms of permanent ground displacement (PGD).

4.1.5 Fault Crossings

Localized permanent ground deformations occur in surface fault rupture areas. Damage to segmented pipes (e.g., cast iron pipe having caulked bell-and-spigot joints) will be heavy when crossing surface ruptured faults. Butt-welded continuous steel pipes may sometimes be able to accommodate fault crossing displacements, up to a few feet.

The amount of fault offset movement is measured in terms of permanent ground displacement (PGD).

Continuous butt-welded steel pipelines are less prone to damage if they are oriented such that tensile strains result from the fault displacement. Tensile deformation takes advantage of the inherent ductility and strength of the steel, while compressive deformation promotes pipe wall wrinkling and the accumulation of high local strain.

The angle of the pipeline-fault crossing has a major impact on pipeline response for orientations that promote tension. For continuous ductile pipelines that cross strike slip faults, performance will improve as the angle of the pipeline-fault intersection increases, in cases where the pipe can displace the surrounding soil consistent with the assumptions outlined by [Newmark and Hall].

For segmented pipelines subject to tension, the optimal angle of the fault crossing depends on joint characteristics. This angle depends upon taking maximum advantage of both the pullout and joint rotational capacities of the joints. Leaded joint couplings appear to be able to take only 1 to 2 inches of fault displacement before failure. Extra-long restrained couplings can take up to about a foot of fault displacement [O’Rourke and Trautmann].

For both segmented and continuous pipelines, it is advantageous to avoid bends, tie-ins and local constraints close to the fault. This allows the pipeline that crosses the fault additional length over which to distribute the imposed strains resulting from the fault offset.

Burial depth is also a factor at fault crossings. The shallower the burial, the less overburdened it will be; hence less frictional resistance by the soil on the pipe. The lower the frictional resistance, the easier the pipe will be able to deform or buckle upwards in fault crossings without causing severe damage. For example, a pipeline with 3 feet of overburden can sustain about four times more fault displacement than a pipeline with 10 feet of overburden.

Two failure modes occur when a pipeline is deformed in compression. The pipeline may buckle as a beam or it may deform by local warping and wrinkling of its wall. Buckling can occur across fault crossings, either due to fault creep or sudden fault offset. Pipe wrinkling failure occurs in thinner walled pipes in high frictionally restraint soil conditions.

4.1.6 Continuous Pipelines

Continuous pipelines are those having rigid joints, such as continuous welded steel pipelines. Built in accordance with modern codes of practice, continuous pipelines have generally performed better in past earthquakes than those constructed using other methods [Newby].

Experience has shown that some pipelines constructed before and during the early 1930s did not benefit from the same quality controls that prevail today. For example, the 1933 Long Beach earthquake caused more than 50 breaks in high-pressure gas pipelines in welded joints. In every instance, the breaks in large diameter lines were discovered at welds that lacked the proper penetration or bond with the body of the pipe. Poor welds have also contributed to failures of 1960s-vintage steel pipelines that were welded using arc-welding procedures.

Experience has also shown that welded pipelines with bends, elbows and local eccentricities will concentrate deformation at these features, especially if permanent ground deformations develop compression strains. Liquefaction-induced landslides during the 1971 San Fernando earthquake caused severe damage to a 49-inch diameter water pipeline at nine bend and welded joints [O'Rourke and Tawfik, 1983].

4.1.7 Segmented Pipelines

A jointed pipeline consists of pipe segments coupled by relatively flexible (or weak) connections (e.g., a bell-and-spigot cast iron piping system). These typically fail in one of three ways: excessive tensile and bending deformations of the pipe barrel, excessive rotation at a joint, or pullout at a joint [Singhal]. Segmented pipe with somewhat rigid caulking such as Portland cement cannot tolerate much movement before leakage occurs. Pipes with flexible rubber gaskets can generally tolerate more seismic deformations.

4.1.8 Appurtenances and Branches

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 or rigid locations that 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 the pipe and the structure. This was reportedly the reason for most of the damage to service connections of water distribution piping systems during the 1971 San Fernando earthquake.

4.1.9 Age and Corrosion

Age and corrosion will accentuate damage, especially in segmented steel, threaded steel and cast iron pipes.

Older pipes appear to have a higher incidence of failure than newer pipes. Pipe damage from the 1987 Whittier Narrows earthquake in the Los Angeles area showed an increasing trend of earthquake pipe breaks versus the age of the pipe [Wang]. Similar trends have been documented for the 1989 Loma Prieta earthquake for steel pipe [Eidinger 1998].

Age effects are possibly strongly correlated with corrosion effects caused by the increasing impact of corrosion over time.

Corrosion weakens pipe by decreasing the material's thickness and by creating stress concentrations. Screwed and threaded steel pipes appear to fail at a higher rate than other types of steel pipes. Some cast iron pipes have also experienced higher incidences of corrosion failure [Isenberg 1978, Isenberg 1979, Isenberg and Taylor].

4.2 General Form of Pipeline Fragility Curves

The damage algorithm for buried pipe is expressed as a repair rate per unit length of pipe, as a function of ground shaking (peak ground velocity, PGV) or ground failure (permanent ground deformation, PGD).

The development of damage algorithms for buried pipe in 2001 is primarily based on empirical evidence, tempered with engineering judgment and sometimes by analytical formulations.

Empirical evidence means the following: after an earthquake, data is collected about how many miles of buried pipe experienced what levels of shaking, and how many pipes were broken or leaking because of that level of shaking.

Most of the empirical evidence prior to 1989 is for the performance of small-diameter (under 12") cast iron pipe. This is because cast iron pipe was the most prevalent material used in water systems for earthquakes that occurred some time ago, such as in San Francisco in 1906. More recent earthquakes like Loma Prieta in 1989 and Northridge in 1994 have yielded new empirical evidence for more modern pipe materials, including asbestos cement, ductile iron and welded steel pipe. However, a complete empirical database for all pipe materials under all levels of shaking still does not exist.

Most documented empirical evidence shows pipe fragility in terms of a repair rate per unit length of pipe. In this report, fragility is described as: repair rate per 1,000 feet of pipe.

For purposes of this report, a pipe repair can be due either to a complete fracture of the pipe, a leak in the pipe or damage to an appurtenance of the pipe. In any case, these repairs require the water agency to perform a repair in the field.

Pipe repairs predicted using the fragility curves are those in buried pipe owned by the water agency. This includes the pipe mains in the street, pipe laterals that branch off the main to fire hydrants and service connections up to the meter owned by the water agency.

Buried pipe from the water agency's meter up to the customer's structure may also break. This pipe is very small in diameter (under 1") and its repair is usually the customer's responsibility. If this pipe breaks, then water will leak out of the water main until someone shuts off the valve at the service connection. Losses due to pipe repairs on the customer's side of the meter are not covered in this report.

4.3 Backbone Pipeline Fragility Curves

Appendix A.1 provides the buried pipeline empirical dataset used for the evaluations presented in Section 4.

Appendices A.2 and A.3 summarize buried pipe earthquake damage statistics and damage algorithms as reported in the literature. Sections 4.3.1 and 4.3.2 compile as much of this previous historical earthquake data as possible into two pipe damage databases: one for wave propagation damage and another for ground failure damage. Statistical analyses are then performed to estimate vulnerability functions.

Vulnerability functions relate overall pipe damage measures to relatively simple demand intensity descriptions. The functions are entirely empirical and are based on reported damage from historical earthquakes. Damage is expressed in terms of pipe repair rate, defined as the number of repairs divided by the pipe length exposed to a particular level of seismic demand. Two separate mechanisms that cause pipe damage are considered: *seismic wave passage* and *earthquake induced ground failure.*

Wave passage effects are transient vibratory soil deformations caused by seismic waves generated during an earthquake. Wave passage effects cover a wide geographic area and affect pipe in all types of soil. Strains are induced in buried pipe because of its restraint within the soil mass. In theory, for vertically propagating shear waves, peak ground strain is directly proportional to peak ground particle velocity (PGV); therefore, PGV is a natural demand description.

Ground failure effects are permanent soil movements caused by such phenomena as liquefaction, lurching, landslides and localized tectonic uplifts. These tend to be fairly localized in a geographic area and potential zones can be identified *a priori* by the specific geotechnical conditions. Ground failure can be very damaging to buried pipe because potentially large, localized deformations can develop as soil masses deform and move relative to each other. Such deformations can cause pipe segments embedded within the soil to fracture or pull out of place. Permanent ground displacement (PGD) is used here as the demand description. The PGD descriptor ignores any variation in the amount of ground displacement and the direction of ground displacement relative to the pipeline. If this level of detail is desired, then site-specific analytical methods should be used instead of area-wide vulnerability functions.

4.3.1 Wave Propagation Damage Database and Vulnerability Functions

The damage considered for the vulnerability functions presented in this section is caused by seismic wave propagation only. The “raw data” damage statistics as reported from various sources are contained in Table A.1-1 and show 164 data points from 18 earthquakes. Many damage statistics cited were in different formats which necessitated adjustments for consistency; this “screened” database is contained in Table A.1-2 and is explained below.

Several aspects of repairs as reported in the damage surveys warrant discussion. The first deals with the accuracy of repair records used as the basis of damage estimation. Typically, detailed damage survey compilations are performed by a third party some time after the water system has been restored. Repair records by field crews are commonly used to ascertain damage counts. The main objective of the field crews is to restore the water system to service as rapidly as possible after the earthquake and, understandably, documenting damage is of secondary importance. As a result, the damage estimates have some inaccuracies, including omitted repair records, vague damage descriptions, and multiple repairs at a single site combined into one record. Unfortunately, this inaccuracy is inherent in all damage surveys, is likely to vary significantly from earthquake to earthquake, and is impossible to quantify.

Life or no differentiation of damage severity is included in the damage surveys. The damage is reported in the survey if, and only if, a repair crew actually performed some type of pipe repair at a particular site. If a repair crew repairs a pipe one day after the earthquake and the same location is repaired again five days after the earthquake, then it is counted as two repairs. (The same pipe can be damaged after it is initially repaired once full system pressures are applied because of continued soil movements). Once the repair crew makes the repair, some type of damage report is developed by the utility. When possible, the reports are reviewed by engineers to decipher the cause and type of damage, but the engineering interpretation may be incorrect since the reports often lack specific information about the damage. For purposes of system-wide hydraulic analysis, it would be useful to differentiate whether the repair was a “small leak” or a “major failure.” A small pipe leak allows continued system operation and thus has relatively low repair priority, while a major failure of pipe segments requires the local system to shut down and no water can flow, which merits a higher repair priority.

The interpretation of repair records to determine the number of damaged pipes varies from earthquake to earthquake, and exactly what was included in the damage counts is not always clear. Repairs can be made to a variety of system components including in-line elements (e.g., pipe, valves, connection hardware) and appurtenances (e.g., service laterals, hydrants or air release valves). Some surveys count damage to in-line elements for use in repair rate calculations; others include damage to utility-owned service laterals up to the utility-customer meter. Still others include damage to service laterals up the customer’s house.

Data on damage only to the main pipe is useful for ascertaining the relative vulnerability of different pipe materials. However, to develop the level-of-effort estimates required to restore the water system to its pre-earthquake condition, all damage requiring field work should be included. Table 4-1 illustrates the effect of counting repairs to customer service laterals, or that portion of service pipe from water main to the customer-utility meter. The surveys suggest that the ratio of service lateral repairs to pipe repairs can vary widely, and the number of service repairs was shown to exceed the numbers of pipe repairs in one of four cases reported. (Note that in Japan, the length of service laterals can be quite long. Typical US water utilities own only a few feet of service lateral up to the meter connection.)

Earthquake	Number of Service Lateral Repairs	Number of Main Pipe Repairs	Ratio Service: Pipe
1995 Hyogoken-nanbu (Kobe) (Shirozu, et al, 1996)	11,800	1,760	6.7:1
1994 Northridge[1] (Toprak, 1998)	208	1,013[2]	1:4.8
1989 Loma Prieta (Eidinger, et al, 1995)	22	113	1:5.1
1971 San Fernando (NOAA, 1973)	557	856	1:1.5

Notes
1. Number of field repair records
2. Includes repairs to hydrants.

Table 4-1. Reported Statistics for Main Pipe and Service Lateral Repairs

If both pipes and service laterals can be repaired during the same site visit, then the service damage counts may not be that important. The opposite is true if each repair requires a separate site visit. Most damage statistics for US earthquakes exclude damage to service laterals on the customer side of the meter, as customers hire private contractors to make service line repairs at the customer's expense. However, in cases where water utility staff repaired a service line on the customer's side of the meter, the damage is included in Tables A.1-1 and A.1-2.

The data in Table 4-1 should be considered as follows. First, calculate the damage to the main pipes using the vulnerability functions presented in Section 4.3.3. Second, allow for an additional 20% in terms of the number of damage locations to account for damage to service laterals, up to the point of the utility-customer meter. Some type of refined analysis for very long service laterals is required if these laterals exceed, on average, about half the width of streets.

The raw data in Table A.1-1 was adjusted and screened to create a database having a consistent format for analysis. The process screened out 83 data points, leaving 81 data points from 12 earthquakes. As shown in Table A.1-2 and summarized in Table 4-2, most of the data points are from the Kobe, Northridge, Loma Prieta and San Fernando earthquakes. For these four earthquakes, respectively, the repair counts were based on the number of repairs to:

- Kobe: in-line components and appurtenances
- Northridge: in-line components and hydrants
- Loma Prieta: in-line components and appurtenances
- San Fernando: in-line components

Earthquake	Data Points	Percentage
1995 Hyogoken-nanbu (Kobe)	9	11%
1994 Northridge	35	43%
1989 Loma Prieta	13	16%
1971 San Fernando	13	16%
Other Earthquakes (8)	11	14%
Total	**81**	**100%**

Table 4-2. Earthquakes and Data Points in Screened PGV Database

The most common material in the database is cast iron (38 data points) followed by steel (13), asbestos cement (10), ductile iron (9), and concrete (2). Another 9 data points represent both cast and ductile iron pipe combined. In terms of pipe diameter, the database contains mostly those sizes associated with distribution main systems; only 8 data points were identified as specifically for large-diameter pipe greater than 12 inches. To consider pipe diameter, refer to Section 4.4.7 for further analysis of the database.

The type of demand used for the study is peak ground velocity (PGV). However, different definitions exist, such as average of the peak horizontal values from orthogonal directions at a point; geometric mean, or square-root of the product of the peak horizontal values; or the peak value from either horizontal direction. Since the intended use of the pipe vulnerability functions is loss estimation from possible future earthquakes, it is natural to base them on the geometric mean PGV since this is the quantity typically estimated using modern attenuation relationships [Sadigh and Egan, 1998]. The geometric mean is usually close to the average and is less than the peak of the two directions.

The Kobe and Northridge data were scaled down to represent the geometric mean PGV values. Scale factors of 0.90 and 0.83, respectively, were determined by averaging numerous Kobe and

Northridge instrument values. Since it was not always clear what was meant by the reported PGV for other earthquakes, inconsistencies in the database are likely. Also, some demands were reported in terms of Modified Mercalli Intensity (MMI) or peak ground acceleration (PGA), and these conversions were made based on Wald et al. [1999]. The variability in PGV values from these different methods is probably moot considering the scatter of repair rate when plotted against PGV, as shown below.

Other adjustments to the raw data include the elimination of data points that were duplicates, contained permanent ground displacement (PGD) effects, or included damage from multiple earthquakes. Errors may be present in the screened database because of misinterpretation of the vague descriptions contained in the sources. Some sources provided multiple damage statistics for the same earthquake and duplicate points were eliminated. Several earthquakes had reported repair rates much greater than the others and the source did not specifically indicate whether PGD effects were present. These were judged to include PGD effects and were eliminated. One earthquake had an aftershock similar in intensity to the main shock, and the repairs for that earthquake were eliminated.

Table A.1-2 contains the screened database that was used for statistical analysis. Data point adjustments are indicated in Table A.1-2 as well.

The database exhibits substantial scatter in plots of repair rate versus PGV. To better discern a causal dependency, PGV ranges were assigned and repair rates were grouped into “bins” according to their associated PGV values. Figure 4-1 shows the median repair rate for each bin. The greater repair rate with increasing PGV suggests that pipe vulnerability functions based on PGV are viable. Two different models were formulated as follows.

Linear (Median) Model. Repair rate RR (repairs per 1,000 feet of pipe), is a straight line function of PGV (inches per sec):

$$RR = a \bullet PGV$$

where

a = the median slope of the data point set, and an individual data point slope is taken as the repair rate divided by its associated PGV. Coefficient a = 0.00187 for the data set having all 81 points. The line defined by this model has the property of having equal numbers of points above and below it. It is one description of central tendency that is not sensitive to data outliers. A two-parameter liner model (RR = 0.01427 + 0.001938 * PGV) has a higher slope, reflecting the influence of the high repair rate of outliers.

Power Model. Repair rate is a function of PGV:

$$RR = b \bullet PGV^c$$

where

b and c = coefficients set using the standard linear least squares method on log (PGV),
b = 0.00108 and c = 1.173 for the data set having all 81 points.

Figure 4-2 shows that both models are about the same, especially when considering the scatter in the data points. Additional analyses were performed to assess the influence of pipe material, pipe diameter and earthquake magnitude. For different pipe materials, relative vulnerability was explored by computing linear models for each material and taking the ratios of the slope coefficients (parameter a). Ductile iron and steel pipe were found to be less vulnerable than cast iron, by less than a factor of two, and asbestos cement was the best performer. These results are not consistent with conventional thinking, which ranks brittle materials such as cast iron or asbestos cement more vulnerable than ductile materials such as steel or ductile iron by more than a factor of three [e.g., NIBS, 1997]. Moreover, statistical tests like the Wilcoxon rank-sum on pairs of material types could not accept the hypothesis at a 5% significance level that the individual data point slope populations differ; an exception was the difference between CI and AC. This suggests that deviations in the linear model slope coefficients from different materials could result from sampling error rather than differing statistical populations.

The models fit the trend of increasing repair rate according to PGV, as suggested by the bin medians also shown. The data point scatter is large, and Figure 4-3 depicts bounds on the variability in terms of 84^{th} and 16^{th} percentile lines constructed so that, respectively, 68 and 13 of the data points fall below the lines. Two-thirds of the points lie between the bounds. The upper bound slope of 0.00529 is 2.8 times the Median Line slope, and the lower bound slope of 0.00052 is 0.28 times the Median Line slope, thus indicating a confidence interval for the vulnerability function. The range is relatively large, having a factor of 10 between the bounds (= 2.8/0.28). If a single lognormal standard deviation were to be applied, beta would be 1.15, or $0.00052 * \exp(2\beta) = 0.00529$).

Additional analyses were performed to assess the influence of pipe material, pipe diameter and earthquake magnitude. For different pipe materials, relative vulnerability was explored by computing linear models for each material and taking the ratios of the slope coefficients (parameter a). Ductile iron and steel pipe were found to be less vulnerable than cast iron, by less than a factor of two, and asbestos cement was the best performer. These results are not consistent with conventional thinking, which ranks brittle materials such as cast iron or asbestos cement more vulnerable than ductile materials such as steel or ductile iron by more than a factor of three [e.g., NIBS, 1997]. Moreover, statistical tests like the Wilcoxon rank-sum on pairs of material types could not accept the hypothesis at a 5% significance level that the individual data point slope populations differ; an exception was the difference between CI and AC. This suggests that deviations in the linear model slope coefficients from different materials could result from sampling error rather than differing statistical populations.

In a similar manner, analyses were carried out to assess the effect of pipe diameter using the dataset in Table A.1-2. With only 8 data points for large-diameter pipe, the results did not show much difference in relative vulnerability versus either distribution pipe or small-diameter pipe. Finally, duration of strong motion shaking during an earthquake could cause cumulative cyclic damage, in which more cycles of deformation lead to more pipe damage. Earthquake magnitude is a surrogate for the duration of strong shaking, but the magnitudes of earthquakes shown in Table A.1-2 are mostly in the range of 6 to 7. No meaningful statistical assessment of a duration effect can be made, even if it is intuitively reasonable to assume that such an effect exists.

Figure 4-4 compares the linear model to several others: HAZUS brittle pipe [NIBS, 1997], Eguchi et al. [1983] cast iron pipe, Eidinger [1998] cast iron pipe, and Toprak [1998] cast iron pipe. The HAZUS model is used in the FEMA US national loss estimation methodology. The

Eguchi model is one of the earliest; it segregated wave propagation from ground failure damage, with demand converted from MMI to PGV using the equation in Wald et al. [1999]. The Toprak model is recent and is based on sophisticated GIS analysis of Northridge pipe damage (based on Northridge data but not as adjusted in the screened database). The linear and Toprak models agree favorably and yield repair predictions less than either the HAZUS, Eidinger or Eguchi models.

4.3.2 PGD Damage Algorithms

The damage considered for the vulnerability functions presented in this section is that caused by permanent ground deformations; wave propagation effects are masked within the more destructive effects of PGDs. The database contains 42 points from four earthquakes, and liquefaction ground failure is the predominate mechanism (Table 4-3).

Earthquake	Number of Data Points	Percentage	Ground Failure Type
1989 Loma Prieta	12	28%	Liquefaction vertical settlement
1983 Nihonkai-Chubu	20[1]	48%	Liquefaction lateral spread
1971 San Fernando	5	12%	Local tectonic uplift
1906 San Francisco	5	12%	Liquefaction lateral spread
Total	**42**	**100%**	
Note 1. Excludes 14 data points for gas pipes listed in database but not used in statistical analysis.			

Table 4-3. Earthquakes and Number of Points in PGD Database

Table A.1-3 contains the complete database. Material types include asbestos cement (20 data points), cast iron (17), and cast iron and steel mixed (5). The diameters are mostly those sizes associated with distribution main systems, with only 5 points specifically identified from large-diameter pipe greater than 12 inches. Table A.1-3 also lists gas pipe damage data that was not used in the statistical analysis. Cast iron gas pipes were reported [Hamada, et al, 1986] to have higher repair rates than the weaker asbestos cement water pipes in the Nihonkai-Chubu quake because gas leaks were detected much more accurately, which implies that many water pipe leaks go undetected. Hamada et al. [1986] did not report the types of joints used in the asbestos cement or steel pipe.

Statistical analysis of the database was conducted in a similar way as that described above for the wave propagation data. Figure 4-5 shows the median repair rates for the different data point bins.

The repair rates are about two orders of magnitude greater than those for wave propagation, thus indicating the extreme hazard that PGD poses for buried pipe. Even for PGDs up to 5 inches, the repair rate is about 2 repairs per 1,000 feet. In the context of post-earthquake water system performance, a system-wide average of only 0.03 “breaks” per 1,000 feet of pipe is assigned a serviceability of 50% using the HAZUS methodology, where 100% serviceability corresponds to the pre-earthquake condition. HAZUS assigns 20% of wave propagation repairs as “leaks”, and 80% of ground failure repairs as “breaks.” Hence, those portions of water systems that experience ground failure are likely to be mostly inoperable immediately after the earthquake. Also, the repair rates are somewhat insensitive to PGD value, as an order of magnitude increase in PGD only produces a factor of roughly 2 to 3 increase in numbers of repairs.

Both Linear and Power models were fitted to the data. The Linear model has coefficient, a = 0.156, and the Power model, b = 1.06, c = 0.319. Figure 4-6 shows that the Power model is a better overall fit to the data. However, for relatively small PGDs that are still quite damaging, the Power model could yield some under-prediction when compared to the median of the data points in this range (see Figure 4-7).

Figure 4-8 depicts bounds on the variability in terms of 84th and 16th percentile curves constructed so that, respectively, 35 and 6 of the data points fall below. About two-thirds of the points lie between the bounds. The upper bound is 2.0 times the Power model, and the lower bound is 0.45 times the Power model, thus indicating a factor of 4.4 times between the 16th and 84th percentiles. If a single lognormal standard deviation were to be applied to the Power model, beta would be 0.74.

Figure 4-9 compares the Power model to HAZUS brittle pipe [NIBS, 1997], Eidinger [1998] for cast iron pipe and the Harding Lawson model for cast iron pipe [Porter et al, 1991]. The HAZUS model is used in the FEMA US national loss estimation methodology. The median Power model yields larger repair rates higher than HAZUS, but lower than the Harding Lawson or Eidinger models.

4.3.3 Recommended Pipe Vulnerability Functions

Table 4-4 provides the recommended "backbone" pipe vulnerability functions (e.g., damage algorithms or fragility curves) for PGV and PGD mechanisms. These functions can be used when there is no knowledge of the pipe materials, joinery, diameter, corrosion status, etc. of the pipe inventory and when the evaluation is for a reasonably large inventory of pipelines comprising a water distribution system.

Hazard	Vulnerability Function	Lognormal Standard Deviation, β	Comment
Wave Propagation	RR=0.00187 * PGV	1.15	Based on 81 data points of which largest percentage (38%) was for CI pipe.
Permanent Ground Deformation	RR=1.06 * PGD$^{0.319}$	0.74	Based on 42 data points of which largest percentage (48%) was for AC pipe.
Notes 1. RR = repairs per 1,000 of main pipe. 2. PGV = peak ground velocity, inches/second. PGD = permanent ground deformation, inches. 3. Ground failure mechanisms used in PGD formulation: Liquefaction (88%); local tectonic uplift (12%).			

Table 4-4. Buried Pipe Vulnerability Functions

4.4 Pipe Damage Algorithms – Considerations for Analysis

The damage algorithms in Table 4-4 can be used to predict damage to buried pipes due to ground shaking, liquefaction and landslide. Table 4-4 should be used in cases where there is no knowledge of pipe materials, pipe joinery, pipe diameter or soil corrosivity. Caution is warranted, however: this practice is akin to using a single damage algorithm for both unreinforced masonry buildings and wood frame buildings, and could produce significantly uncertain results or be unsuitable for loss estimation purposes. For these reasons, more refined damage algorithms are included in the following sections.

4.4.1 Fragility Curve Modification Factors

The fragility curves in Table 4-4 are "backbone" fragility curves representing the average performance of all kinds of pipes in earthquakes. Throughout this report and in Appendix A, the possible behavior of various pipe types in earthquakes is addressed. Tables 4-5 and 4-6 present summary recommendations on how to apply the fragility curves in Table 4-4 to particular pipe types. By diameter, small means 4 to 12 inches in diameter, and large means 16 inches in diameter and larger. Tables 4-5 and 4-6 are for pipelines installed without seismic design specific to the local geologic conditions.

Pipe Material	Joint Type	Soils	Diam.	K_1	Reference Sections
Cast iron	Cement	All	Small	1.0	4.4.2
Cast iron	Cement	Corrosive	Small	1.4	4.4.2
Cast iron	Cement	Non-corrosive	Small	0.7	4.4.2
Cast iron	Rubber gasket	All	Small	0.8	4.4.2
Welded steel	Lap - Arc welded	All	Small	0.6	4.4.4
Welded steel	Lap - Arc welded	Corrosive	Small	0.9	4.4.4
Welded steel	Lap - Arc welded	Non-corrosive	Small	0.3	4.4.4
Welded steel	Lap - Arc welded	All	Large	0.15	4.4.4
Welded steel	Rubber gasket	All	Small	0.7	4.4.6
Welded steel	Screwed	All	Small	1.3	4.4.6 A.3.11
Welded steel	Riveted	All	Small	1.3	4.4.6
Asbestos cement	Rubber gasket	All	Small	0.5	4.4.3 4.4.5
Asbestos cement	Cement	All	Small	1.0	4.4.3
Concrete w/Stl Cyl.	Lap - Arc Welded	All	Large	0.7	4.4.6
Concrete w/Stl Cyl.	Cement	All	Large	1.0	4.4.6
Concrete w/Stl Cyl.	Rubber Gasket	All	Large	0.8	4.4.6
PVC	Rubber gasket	All	Small	0.5	4.4.6
Ductile iron	Rubber gasket	All	Small	0.5	4.4.5 4.4.6

Table 4-5. Ground Shaking - Constants for Fragility Curve

Pipe Material	**Joint Type**	**K_2**	**Reference Sections**
Cast iron	Cement	1.0	4.4.2
Cast iron	Rubber gasket	0.8	4.4.2
Cast iron	Mechanical restrained	0.7	4.4.2
Welded steel	Arc welded, lap welds (large diameter, non corrosive)	0.15	4.4.4
Welded steel	Rubber gasket	0.7	4.4.3
Asbestos cement	Rubber gasket	0.8	4.4.3
Asbestos cement	Cement	1.0	4.4.6
Concrete w/Stl Cyl.	Welded	0.6	4.4.6
Concrete w/Stl Cyl.	Cement	1.0	4.4.6
Concrete w/Stl Cyl.	Rubber Gasket	0.7	4.4.6
PVC	Rubber gasket	0.8	4.4.6
Ductile iron	Rubber gasket	0.5	4.4.6

Table 4-6. Permanent Ground Deformations - Constants for Fragility Curve

To apply Tables 4-5 and 4-6, the pipe vulnerability functions in Table 4-4 are adjusted as follows:

$$RR = K_1(0.00187)PGV \text{ (for wave propagation)}$$

$$RR = K_2(1.06)PGD^{0.319} \text{ (for permanent ground deformation)}$$

4.4.2 Cast Iron Pipe Fragility Curve

The cast iron pipe fragility curve should include the following considerations:

- If the cast iron pipe is located in soils with uncertain corrosive soil conditions, set K1 = K2 = 1.0. This reflects that the bulk of the empirical data set is governed by cast iron pipe with either cement or lead-type joints.
- If the cast iron pipe is in corrosive soils, the damage rate should be higher than if the pipe is in non-corrosive soils. Unfortunately, the bulk of the empirical database does not provide information on soil corrosiveness. Engineering judgment says that a small-diameter cast iron pipe in corrosive soil is about 40% more susceptible to damage than the best fit curve from the empirical database, and that cast iron pipe in non-corrosive soils is about 30% less susceptible to damage than the best fit curve from the empirical database. This translates to a factor of 2 difference between cast iron pipe in corrosive versus non-corrosive soils (1.4/0.7 = 2.0).
- If the cast iron pipe uses rubber gasket joints like those occasionally used by some water utilities, assume about 80% of the damage rate for ground shaking and about 80% of the damage rate for ground deformation. This reflects that gasketed pipe of all types, including AC and DI, have lower damage rates than cement or lead-jointed cast iron pipe, and factors in the relative earthquake vulnerability of rubber gasketed cast iron pipe that is suggested in Table A.3-18.

- The K1 constants in Table 4-5 can be multiplied by 0.5 for cast iron pipe with a 16-inch diameter and larger.
- K2 for restrained CI pipe is set about 30% lower than regular cemented joint CI pipe. A limited length of restrained CI pipe is in use, so there is no empirical data available to confirm this trend. Based on engineering judgment, the restraint offered by bolted joints should provide some extra ability of CI pipe to sustain PGD before being damaged.

4.4.3 Asbestos Cement Pipe

The asbestos cement pipe corrections factors K1 and K2 include the following considerations:

- The Loma Prieta earthquake showed that AC pipe in the epicentral area of the earthquake with rubber gasketed joints and 8-foot to 13-foot pipe segments had better seismic performance than would have been anticipated by using older empirical models (see Figure A-3), at least in areas subject only to ground shaking.
- The empirical data for rubber gasketed asbestos cement pipe in the 1989 Loma Prieta and 1994 Northridge earthquakes differs considerably from previously reported empirical data for asbestos cement pipe in the Haicheng or Mexico City earthquakes [O'Rourke and Ayala]. One explanation of the AC pipe damage in those earthquakes is that cemented joints were used predominantly instead of rubber gasketed joints. Cemented joints limit the flexibility of the pipe. This factor is considered in differentiating the damage algorithm for AC pipe into two: one for rubber gasketed pipe, which is better than cast iron pipe; and one for cemented joint pipe, which shows similar performance as the cast iron pipe.
- AC pipe in areas subject to settlements (PGDs). have had high damage rates (e.g., Turkey, 1999). The K2 factors of 1.0 for cemented joints or 0.8 for rubber gasketed joints reflect little reduction from the backbone fragility curve for AC pipe.

4.4.4 Welded Steel Pipe

The welded steel pipe fragility curve should include the following considerations:

- If the steel pipe is in corrosive soils, the damage rate should be higher than if the pipe is in non-corrosive soils. Engineering judgment indicates that a small-diameter steel pipe in corrosive soil is about 50% more susceptible to damage than the best fit curve from the empirical database, and that small-diameter steel pipe in non-corrosive soils is about 50% less susceptible to damage than the best fit curve from the empirical database. This translates to a factor of 3 difference between welded steel pipe in corrosive versus non-corrosive soils (1.5/0.50 = 3.0). Adjustment for corrosion should be applied only when no corrosion protection measures have been taken and the pipe is in corrosive or moist soil. Corrosion measures might include a suitable coating system with sacrificial anodes.
- Note that for steel pipe with corrosion protection—including suitable coating and sacrificial anodes or suitable coating with impressed current—the use of correction factors for corrosion may not be suitable.

- Corrosion is an age-related phenomenon. Relatively new steel pipe (under 25 years old) in corrosive soil environments will not be as affected by corrosion as older steel pipe (more than 50 years old) in the same environment. Similarly, corrosion will not play as big a role if special corrosion protection is included in the design. For these cases, use K1 = 0.3 for small-diameter welded steel pipe.

- The factor of 3 increase in repair rates is representative of corroded pipe based on the 1971 San Fernando, 1983 Coalinga and 1989 Loma Prieta earthquake experiences.

- If age is not an attribute that can be determined for a limited effort loss estimation study, an average corrosion factor of 2 should be used when steel pipe is located in corrosive soils. For this case, use K1 = 0.6 for small-diameter welded steel pipe.

- The repair rates are decreased for steel pipe having nominal diameters greater than or equal to 12 inches. The 1989 Loma Prieta empirical evidence indicates a repair rate diameter dependency [Eidinger, 1998]. Other studies [Sato and Myurata, O'Rourke and Jeon] also report lower damage rates for large-diameter pipes. Important factors may include the quality of construction, fewer lateral connections and alignments possibly in better soils. Considering these factors, a diameter dependency for large-diameter pipes shows that repair rates are reduced by 75%. The reduction in repair rates for large-diameter pipe probably reflects a number of factors:

  - Few service connections are attached to large diameter pipe.
  - Corrosion effects on large-diameter pipes, which can lead to small pin hole leaks, are not as pervasive for large-diameter pipes as for small-diameter pipes.
  - There are fewer bends and tees in large-diameter pipes (e.g., stress risers).
  - Large-diameter pipes have thicker walls to contain an equal amount of pressure and are therefore stronger.
  - Large-diameter pipes may be installed with better care.
  - It is easier to weld large-diameter pipes than small-diameter pipes.
  - Soil loads, as a function of pipe strength, are lower for large-diameter pipe given the same depth of soil cover.

4.4.5 Compare Cast Iron, Asbestos Cement and Ductile Iron Pipe

The curves in Figure 4-10 represent the best fit lines through the empirical database only for small-diameter cast iron, ductile iron and asbestos cement pipe for wave propagation damage from the 1994 Northridge earthquake. The following observations are made:

- Ductile iron pipe has the lowest damage rates at the lowest PGVs.

- AC pipe has similar damage rates as DI pipe and has the lowest damage rate at PGVs over 14 inches/second.

- Cast iron pipe has the highest damage rates.

Based on the complete data set in Table A.1-2, vulnerability functions are fitted through data for specific types of pipe. The following models are found for pipe damage due to ground shaking:

- Cast iron pipe. RR=0.00195 * PGV. Damage rates are 104% (=195/187) of the average. (RR = 0.00195 * PGV for cast iron pipe is based on only CI data points).
- Ductile iron pipe. RR=0.00103 * PGV. Damage rates are 55% (=103/187) of the average. (RR = 0.00103 * PGV for DI pipe is based on only DI data points).
- Asbestos cement pipe. RR=0.00075 * PGV. Damage rates are 40% (=75/187) of the average. (RR = 0.00075 * PGV for AC pipe is based on only AC data points).

4.4.6 Other Pipe Materials

Insufficient empirical evidence currently exists to describe performance for many classes of buried pipe. For example, an earthquake has not yet occurred that has severely tested large quantities of PVC pipe with rubber-gasketed joints. Since many water systems have this type of pipe in the ground, recommendations for treating the various classes of pipe are.

- **Ductile Iron.** Use the cast iron damage algorithm for unknown soil conditions, but scaled by about 0.50, based on the empirical evidence in the 1994 Northridge earthquake. Note that some ductile iron pipe networks include cast iron appurtenances, making them the weak link.
- **Welded Steel Arc Welded X Grade.** By X grade, it is meant welded steel pipelines installed to the general quality controls and design procedures commonly used for oil and gas pipelines. Joints are generally butt-welded. Use the cast iron damage algorithm for unknown soil conditions, but scaled by 0.01, based on algorithms reported in the literature (e.g., Figure A-3).
- **Concrete with Steel Cylinder.** These are generally large diameter pipes, typically 24" to 60" in diameter. Three typical pipe joints are used: lap welds of the internal steel cylinder; cemented joints, and carnegie (e.g., rubber gasket) joints. The thin wall of the internal steel cylinder is usually designed to take between one-third and two-thirds of the hoop tension. The limited data available for this type of pipe, coupled with the thin wall and eccentric welds of the internal cylinder, suggest a base rate curve about equal to the average of the empirical data set. Table A.3-18 suggests that the relative vulnerability for these kinds of pipe is about 12 for gasketed joints to about 14 for welded joints. If K1 = 1.0 for cast iron pipe, then K1 = 0.5 or so is suggested for these kinds of pipe. Allowing for the lack of empirical evidence available at this time, and noting that at least one 60-inch diameter pipe failed in the 1989 Loma Prieta earthquake at low g levels, it is difficult to establish K1 or K2 constants with much certainty. The approach in this report is to set K1 and K2 as somewhat lower than 1.0, but not as low as suggested by Table A.3-18.
- **Riveted Steel.** Use about two times the arc-welded steel damage algorithm.
- **Steel, Rubber Gasket.** Use the arc-welded steel damage algorithm scaled by 1.2.

- **PVC, Rubber Gasket.** Use the asbestos cement damage algorithm (e.g., rubber gasket). The rationale is that segmented pipe having similar joint qualities should have similar seismic performance. Engineering judgment indicates that plastic PVC pipe with rubber-gasketed joints is somewhat better than similar AC pipe, due to plastic's better tensile strength capability, but is somewhat worse than AC pipe because of longer segment sections, thereby increasing joint pullout demands. Lacking empirical evidence, equivalent pipe properties are assumed. In practice, the relative capacity of rubber-gasketed AC versus PVC pipe is likely to be strongly correlated to the relative insertion depths for the specific installations, in that a shorter installation depth leads to a weaker pipe.

4.4.7 Effect of Pipeline Diameter

Various researchers over the past 20 years have considered that the diameter of the pipe has some bearing on the capacity of the pipe to withstand the effects of earthquakes without damage. For example:

- Section A.3.1 (Memphis study) suggests fragility curves that have a constant varying from 1.0 to 0.0 as pipeline diameter increases from 4 inches to more than 40 inches.
- Section A.3.11 (Loma Prieta) includes empirical evidence (Figure A-11) showing a reduction in damage rates for larger diameter welded steel pipe, but no such clear indication for cast iron or asbestos cement pipe.
- Appendix G (Northridge) includes empirical evidence showing a reduction in damage rates for cast iron, asbestos cement and ductile iron pipes, with increasing diameter.

The strong trend of bigger diameter equaling much lower damage rates, as shown in the Northridge data for cast iron pipe, is not indicated in the Loma Prieta data, in which bigger diameter equals about the same damage rate and, possibly, a slight decrease. The Loma Prieta data also shows an increasing damage rate with increasing diameter for asbestos cement pipe. The question is: Why? How should the fragility curves account for this behavior? The answers may lie in an explanation based on strength of mechanics principles. Section A.3.11 provides some suggestions.

A possible reason why small-diameter pipe has shown higher damage rates in at least some earthquakes is that they were located in the worst soil areas and were constructed with the lowest quality control. If these explanations are true, then the diameter effect seen in the Northridge data set may not be true for other water systems.

Tables 4-7 and 4-8 present damage data for the combined cities of Kobe, Ashiya and Nishinomiya for the 1995 Kobe earthquake [after Shirozu et al]. These tables suggest no particular diameter dependency for common diameter distribution pipes of 4" to 12" diameter; a higher rate for very small diameter pipe of less than or equal to 3" diameter, which is uncommon in the US except for service laterals; and a moderately lower damage rate for large-diameter pipes of 16" diameter and larger. Tables 4-7 and 4-8 make no distinction between pipe diameter versus level or type of seismic hazard, so it is possible that large-diameter pipes were located in areas with less shaking or less ground failure. For Table 4-8, the total number of repairs was 915 for ductile iron pipe and 611 for cast iron pipe.

Pipe Diameter	Repairs	Length (km)	Repair Rate per km
≤ 75 mm	505	266.1	1.898
100 – 150 mm	1,317	1,423	0.926
200 – 250 mm	412	439.9	0.937
300 – 450 mm	283	362.6	0.783
≥ 500 mm	87	169.5	0.513

Table 4-7. Pipe Repair, 1995 Kobe Earthquake, By Diameter, All Pipe Materials

Pipe Diameter	Repair Rate per km, Cast Iron Pipe	Repair Rate per km, Ductile Iron Pipe	Ratio, DI to CI
≤ 75 mm	2.600	1.029	0.40
100 – 150 mm	1.860	0.486	0.26
200 – 250 mm	1.687	0.545	0.32
300 – 450 mm	0.850	0.480	0.56
≥ 500 mm	0.301	0.061	0.20

Table 4-8. Pipe Repair, 1995 Kobe Earthquake, By Diameter, CI and DI Pipe

In conclusion, there is not enough empirical evidence to prove a diameter effect exists for all pipe materials in any given water system. However, the empirical evidence strongly indicates that some relationship does exist, and that the largest pipes, those over 12" diameter, have lower damage rates than do common diameter distribution pipes of 4” to 12” diameter.

4.5 Fault Crossing Pipe Damage Algorithms

For fault crossings, the amount of offset and the pipe material are critical parameters in determining whether the pipeline will break. Other parameters such as soil backfill, angle of pipeline crossing, depth of burial are also important.

A simple vulnerability model is proposed as follows:

- Determine the mean amount of fault offset along the entire length of the fault.
- Damage algorithm:
  - Continuous pipeline (e.g., welded steel):

    $P_{\text{no failure}} = 1 - 0.70 * \frac{PGD}{60}$, $\quad P_{\text{no failure}} \bullet 0.05$

  - Segmented pipeline (e.g., cast iron with cemented joints):

    $P_{\text{no failure}} = 1.00$, PGD = 0
    $P_{\text{no failure}} = 0.50$, PGD = 1 to 12 inches
    $P_{\text{no failure}} = 0.20$, PGD = 13 to 24 inches
    $P_{\text{no failure}} = 0.05$, PGD = over 24 inches

This simple vulnerability model should be used only for vulnerability analyses of a large inventory of pipelines that cross faults. For pipe-fault-specific conditions, analytical techniques such as those described in the *ASCE Guidelines for the Seismic Design of Oil and Gas Pipeline Systems* [1984] can be used to evaluate pipe-specific performance.

4.6 Other Considerations

4.6.1 Single Pipeline Failure Algorithm

To obtain a probability of failure for an individual pipeline link of length L, a Poisson probability distribution is used:

$$P(x=k) = (\lambda L)^k e^{-\lambda L}/k!$$

where

x is a random variable denoting the number of times the event of a broken pipe occurs, λ is the rate at which the event occurs, and λL is the average number of occurrences occurring over length L of pipe that is being examined. λ is determined as the highest value from the wave propagation and permanent ground deformation models described in prior sections.

Since a single break in a pipe places the entire pipeline length out of service, the probability of service for an individual pipeline can be easily calculated by setting (k=0). For simplicity, assume that only break pipe repairs will put a pipeline out of service.

$$P_{\text{pipeline link i in service}} = - e^{-\lambda L} = P_i$$

For a pipeline (named j) composed of many individual pipe links (P_i), the probability that the pipeline will not deliver flow through its entire length will be 1 minus the probability that all single links are in service. Thus:

$$P_{\text{pipeline j out of service}} = 1 - \prod_{i=1}^{n} P_i$$

4.6.2 Variability in Results

The results presented in Section 4 show widespread scatter in the track record of buried pipe performance in past earthquakes. Considerable uncertainty and randomness must be addressed in the development of pipeline fragility curves. Table 4-4 provides the lognormal standard deviations for the backbone fragility curves; these values are large, in part because the backbone fragility curves combine all empirical data for different pipe material and other conditions. In the following paragraphs, sources of this variability are addressed along with recommendations for using lognormal standard deviations when the backbone fragility curves of Table 4-4 are combined with the pipe-specific factors noted in Tables 4-5 and 4-6.

In this report, damage algorithms use the commonly used damage “measurement” of repair rate per 1,000 feet. This is a measure of an overall or global description of pipe damage. The variability in pipe performance is accounted for by incorporating uncertainty and randomness. Randomness is incorporated according to whether a particular geologic hazard will occur. Given that the hazard occurs, uncertainty is incorporated according to whether a particular pipe will fail.

Randomness in the ground motion can be accounted by calculating the damage to the entire buried pipe system for the median ground motion hazards, recognizing that the response of individual pipes may vary because of random differences in the local ground motions. When evaluating a large population of pipe (e.g., more than 1,000 miles), randomness in ground motions at one point generally are counterbalanced by randomness in ground motion at another location, and the effects tend to cancel each other out. However, even with large populations of pipe, randomness remains; the hazard attenuation model for a particular earthquake may not actually match the scatter in the observed motions.

Based on the Table 4-4 backbone fragility curves and when applied to specific types of pipe using the modification factors in Tables 4-5 and 4-6, the results should be considered to be accurate only within ±50% of the predicted damage. These ranges reflect about a 67% probability that the actual pipe damage will be within these bounds. This uncertainty band reflects a β of about 0.40. When evaluating a small population of pipe (e.g., less than 10 miles), randomness in ground motions is an important factor, and the results should be considered to be accurate only within ±60% of the predicted damage. These ranges reflect about a 67% probability that the actual pipe damage will be within these bounds. This uncertainty band reflects a β of about 0.56.

This variability is due, in part, to the following factors:

- Previous studies of past earthquakes have yielded repair rate data based upon limited assessments. Many times large areas were assessed a single MMI value, ignoring microzonation issues, and the actual mileages of pipe, pipe type and level of shaking or induced permanent ground deformations were estimated. Detailed attributes for every pipe—materials, corrosive soil conditions and type of joints—often were not tabulated accurately. Databases with detailed reviews of pipe damage are relatively limited.

- Some of the data is from earthquakes causing relatively moderate levels of shaking. Some of the earthquakes created shaking levels in the range of 0.10 to 0.30 g peak ground accelerations in areas that suffered the most pipe damage. At these levels of shaking, most pipes actually are *not* damaged; a reported repair rate of one per 1,000 feet, which is a high repair rate, actually means that only 1 in 83 12-foot-long pipe segments actually failed. In other words, about 99% of the pipes are undamaged. The empirical evidence is mostly for repair rates in the “tails” of the pipe’s fragility distribution.

- Intensity data, such as Modified Mercalli Intensity (MMI) for historical earthquakes, is imperfect. For example, two different investigators can assign different intensities to the same data, as evidenced by assignments of MMI VIII or X to the city of Coalinga, as affected by the 1983 Coalinga earthquake [Hopper et al, Thiel and Zsutty]. The same is true for other measures of seismic intensity such as peak ground acceleration, velocity or vertical or lateral ground movements, particularly when these have been inferred based on MMI data and not on instrumental recordings.

- Prior to widespread use of GIS systems around 1990, estimates on the length of pipeline exposed to an earthquake were often approximate with respect to materials, joints and total lengths of pipe. Very little data is available that relates pipelines age or soil corrosivity to levels of damage.
- Repair data have often been reconstructed from the memory of workers or from incomplete or inaccurate data sets.
- One repair can be associated with several leaks in the same pipe barrel; conversely, one break can lead to several repairs.
- Although data on certain pipe materials can be complete at specific MMI intensities, it is not usually complete for all pipe materials at all intensities. Certain pipe materials and/or joint units have very limited actual earthquake performance data. The quality of construction can vary among different countries or cities from which the earthquake data was obtained.
- Repair data for some earthquake events represent data for limited lengths of pipe in high intensity shaken areas, which can result in misleading damage trends because of the small sample size effects.
- Repair data available from the literature often incorporates damage from ground shaking (e.g., wave propagation), as well as ground movements (e.g., surface faulting, liquefaction or landslides). The quality of the process of unaggregating this data into components directly attributable to one type of earthquake hazard introduces uncertainty into the resulting data.

Such variabilities mean that judgment must be used in applying this empirical data in the development of damage algorithms. This report relies more heavily on the well-documented 1989 Loma Prieta damage to EBMUD’s system and 1994 Northridge damage to LADWP’s system than on older empirical data sets. Undoubtedly, as new empirical data sets become available, improvements in pipe damage algorithms will be possible.

4.7 References

ASCE, 1984, *Guidelines for the Seismic Design of Oil and Gas Pipeline Systems*, prepared by the ASCE Technical Council on Lifeline Earthquake Engineering, D. Nyman, Principal Investigator, 1984.

Eguchi, R., T., Legg, M.R., Taylor, C. E., Philipson, L. L., Wiggins, J. H., *Earthquake Performance of Water and Natural Gas Supply System*, J. H. Wiggins Company, NSF Grant PFR-8005083, Report 83-1396-5, July 1983.

Eidinger, J. "Lifelines, Water Distribution System," in *The Loma Prieta, California, Earthquake of October 17, 1989, Performance of the Built Environment -Lifelines*, US Geological Survey Professional Paper 1552-A, pp A63-A80, A. Schiff Ed., Dec. 1998.

Hamada, M., Yasuda, S., Isoyama, R., and Emoto, K., *Study on Liquefaction Induced Permanent Ground Displacements*, Japan Association for the Development of Earthquake Prediction, Nov. 1, 1986.

Hopper, M. G., Thenhaus, P.C., Barnhard, L., Algermissen, S.T. "Damage Survey in Coalinga, California, for the Earthquake of May 2, 1983," *Coalinga California, Earthquake of May 2, 1983, Special Publication 66*. Sacramento, California, California Division of Mines and Geology, 1984.

Isenberg, J., *Seismic Performance of Underground Water Pipelines in the Southeast San Fernando Valley in the 1971 San Fernando Earthquake*, Grant Report No. 8, Weidlinger Associates, New York, New York, Sept. 1978.

Isenberg, J., "Role of Corrosion in Water Pipeline Performance in Three U.S. Earthquakes," *Proceedings, 2nd US National Conference on Earthquake Engineering*, Stanford, California, Aug. 1979.

Isenberg, J., Taylor, C. E., "Performance of Water and Sewer Lifelines in the May 2, 1983, Coalinga California Earthquake," *Lifeline Earthquake Engineering, Performance, Design and Construction*, ASCE, New York, New York, Oct. 1984.

Newby, A. B., "Pipelines Ride the Shockwaves," *Proceedings 45*, Pacific Coast Gas Association, San Francisco, California, 1954.

Newmark, N.M., and Hall, W. J., "Pipeline Design to Resist Large Fault Displacement," presented at the US National Conference of Earthquake Engineering, Ann Arbor, Michigan, June 1975.

NIBS, *Earthquake Loss Estimation Methodology HAZUS 97: Technical Manual*, Federal Emergency Management Agency, Washington, D.C., Vol. 2, 1997.

O'Rourke, M.J., and Ayala, G., 1993, "Pipeline Damage to Wave Propagation," *Journal of Geotechnical Engineering*, ASCE, Vol. 119, No. 9, 1993.

O'Rourke, M., and Nordberg, C., "Analysis Procedures For Buried Pipelines Subject to Longitudinal and Transverse Permanent Ground Deformation," *Proceedings—3rd US-Japan*

Workshop on Earthquake Resistant Design of Lifeline Facilities and Countermeasures for Soil Liquefaction.

O'Rourke, T.D., and Jeon, S-S., "Factors Affecting the Earthquake Damage of Water Distribution Systems, in Optimizing Post-Earthquake Lifeline System Reliability," *TCLEE Monograph No. 16*, ASCE, 1999.

O'Rourke, T. D., and Tawfik, M.S. "Effects of Lateral Spreading on Buried Pipelines During the 1971 San Fernando Earthquake," *Earthquake Behavior and Safety of Oil and Gas Storage Facilities, Buried Pipelines and Equipment*, PVC, Vol.77, New York, New York, ASME, 1983.

O'Rourke, T. D., and Trautmann, C. H., "Earthquake Ground Rupture Effects on Jointed Pipe," *Proceedings on the 2nd Specialty Conference of the Technical Council on Lifeline Earthquake Engineering*, ASCE, August, 1981.

Porter, K.A., Scawthorn C., Honegger, D. G., O'Rourke, T.D., and Blackburn, F., "Performance of Water Supply Pipelines in Liquefied Soil," *Proceedings—4th US-Japan Workshop on Earthquake Disaster Prevention for Lifeline Systems*, NIST Special Publication 840, August 1992.

Sadigh, R.K., and Egan, J.A., "Updated Relationships For Horizontal Peak Ground Velocity and Peak Ground Displacement For Shallow Crustal Earthquakes," *Proceedings of the 6th U.S. National Conference on Earthquake Engineering*, EERI, Seattle, June 1998.

Sato, Ryo and Myurata, Masahiko, *GIS-Based Interactive and Graphic Computer System to Evaluate Seismic Risks on Water Delivery Networks*, Dept. of Civil Engineering and Operations Research, Princeton University, Dec. 1, 1990.

Shirozu, T., Yune, S., Isoyama, R., and Iwamoto, T., "Report on Damage to Water Distribution Pipes Caused by the 1995 Hyogoken-Nanbu (Kobe) Earthquake," *Proceedings of the 6th Japan-U.S. Workshop on Earthquake Resistant Design of Lifeline Facilities and Countermeasures Against Soil Liquefaction*, National Center for Earthquake Engineering Research Report NCEER-96-0012, Sept. 1996.

Singhal, A. C., "Nonlinear Behavior of Ductile Iron Pipeline Joints," *Journal of Technical Topics in Civil Engineering*, 110, No. 1., ASCE, May 1984.

Thiel, C.C. Jr., Zsutty, T.C., *Earthquake Parameters and Damage Statistics*, San Francisco, Forrell/Elsessor Engineers, 1987.

Toprak, Selcuk, *Earthquake effects on buried pipeline systems*, Ph.D. Dissertation, Cornell University, Aug. 1998.

Wald, D.J., Quitoriano, V., Heaton, T.H., and Kanamori, H., "Relationships between Peak Ground Acceleration, Peak Ground Velocity, and Modified Mercalli Intensity in California," *Earthquake Spectra*, Vol. 15, No. 3, Aug. 1999.

Wang, L., *A New Look Into the Performance of Water Pipeline Systems From 1987 Whittier Narrows, California Earthquake*, Department of Civil Engineering, Old Dominion University, No. ODU LEE-05, Jan. 1990.

4.8 Figures

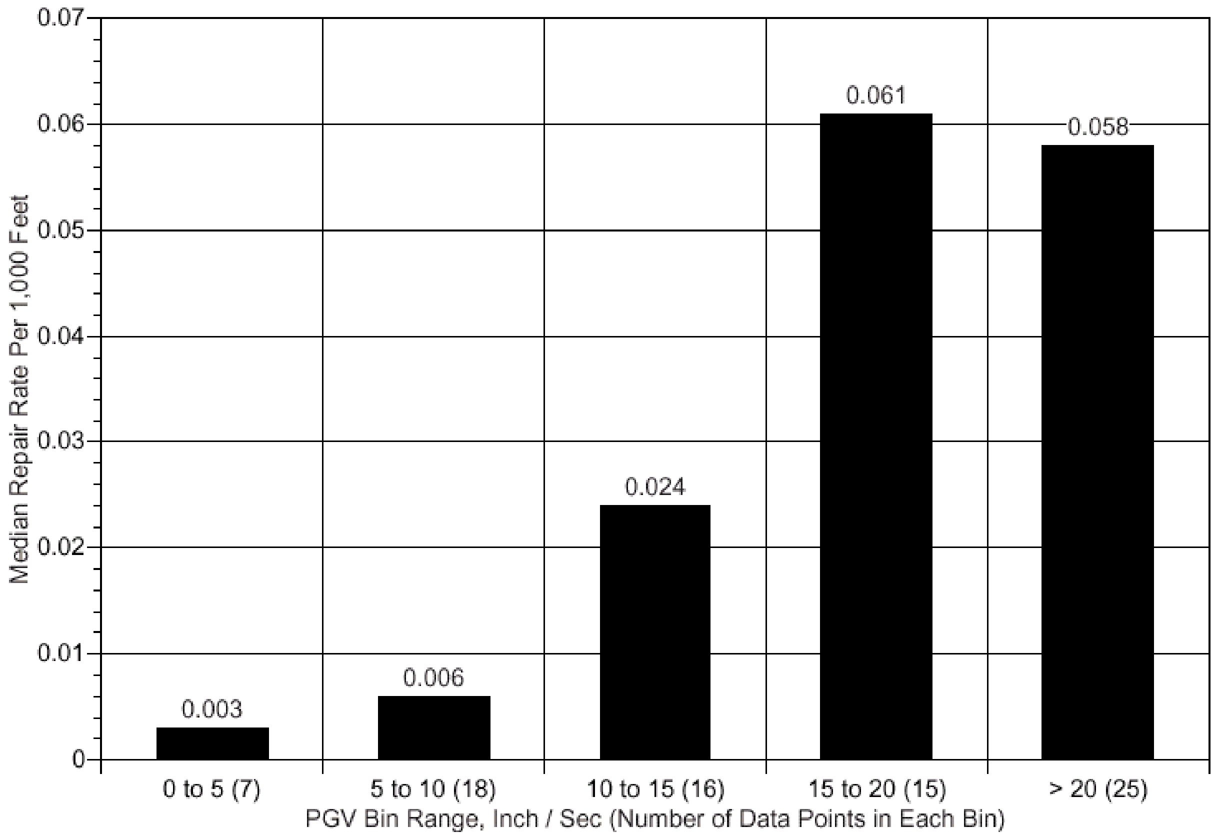


Figure 4-1. Bin Median Values (Wave Propagation)

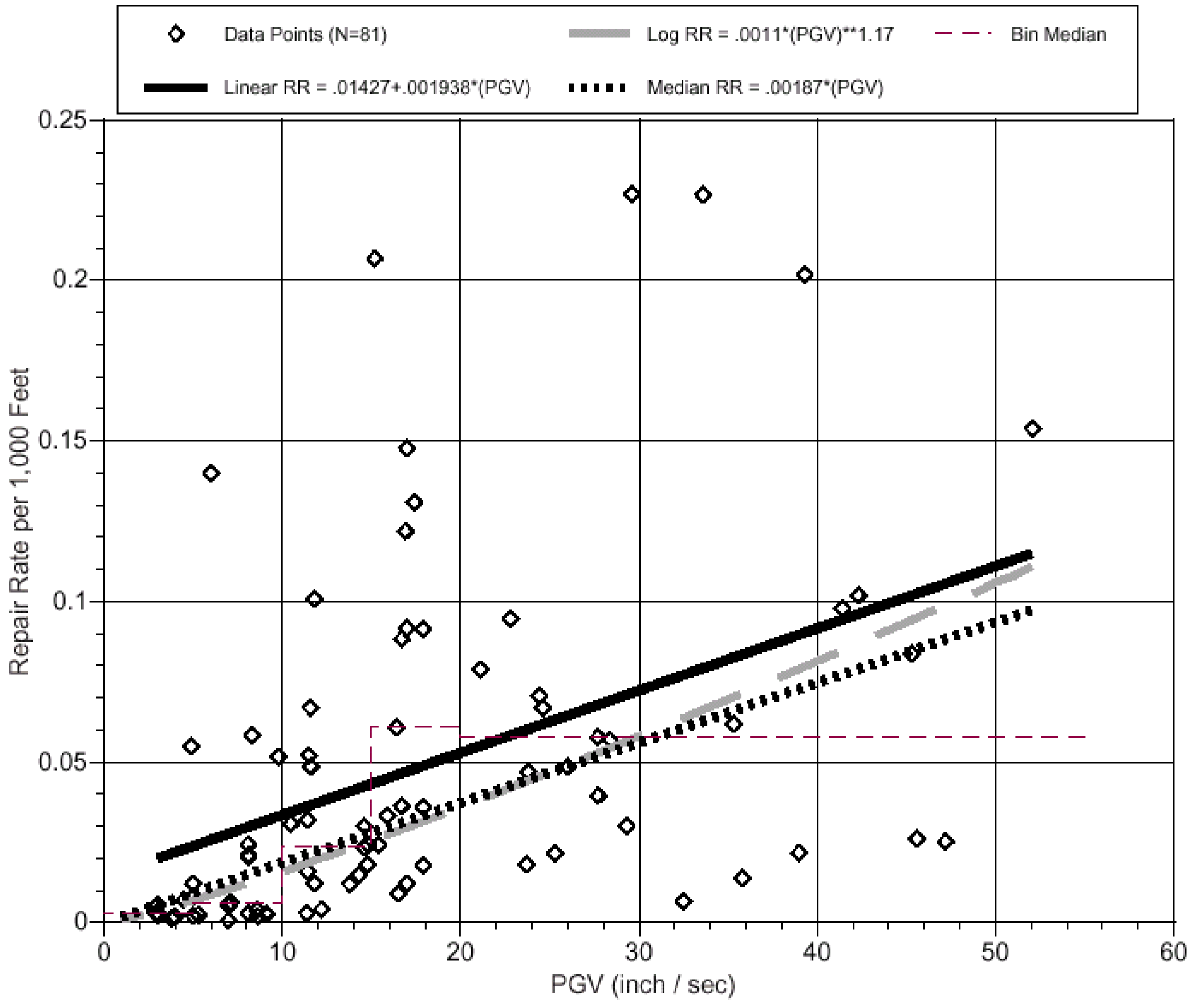


Figure 4-2. Vulnerability Functions (Wave Propagation)

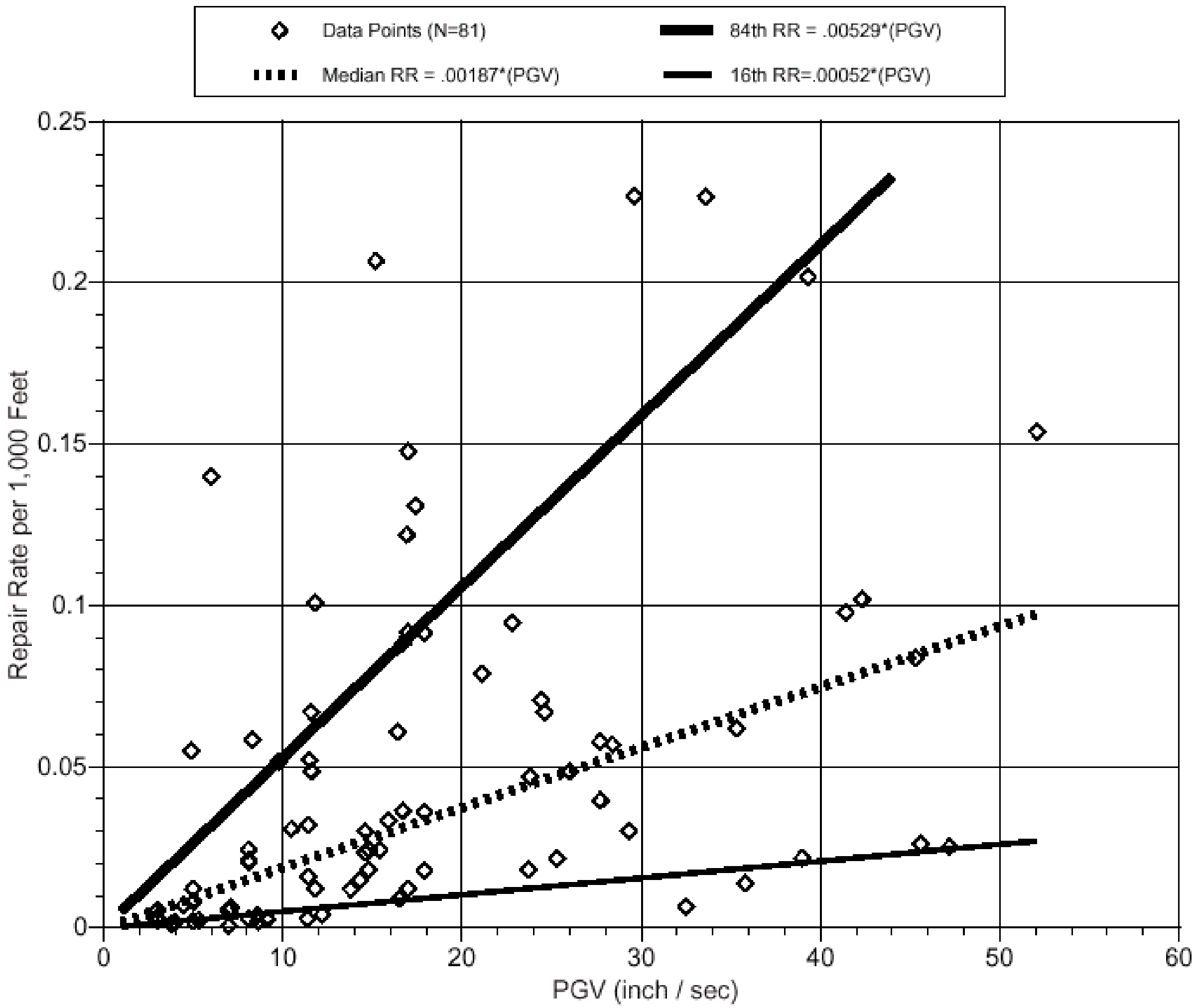


Figure 4-3. Median, 84th and 16th Percentile Functions (Wave Propagation)

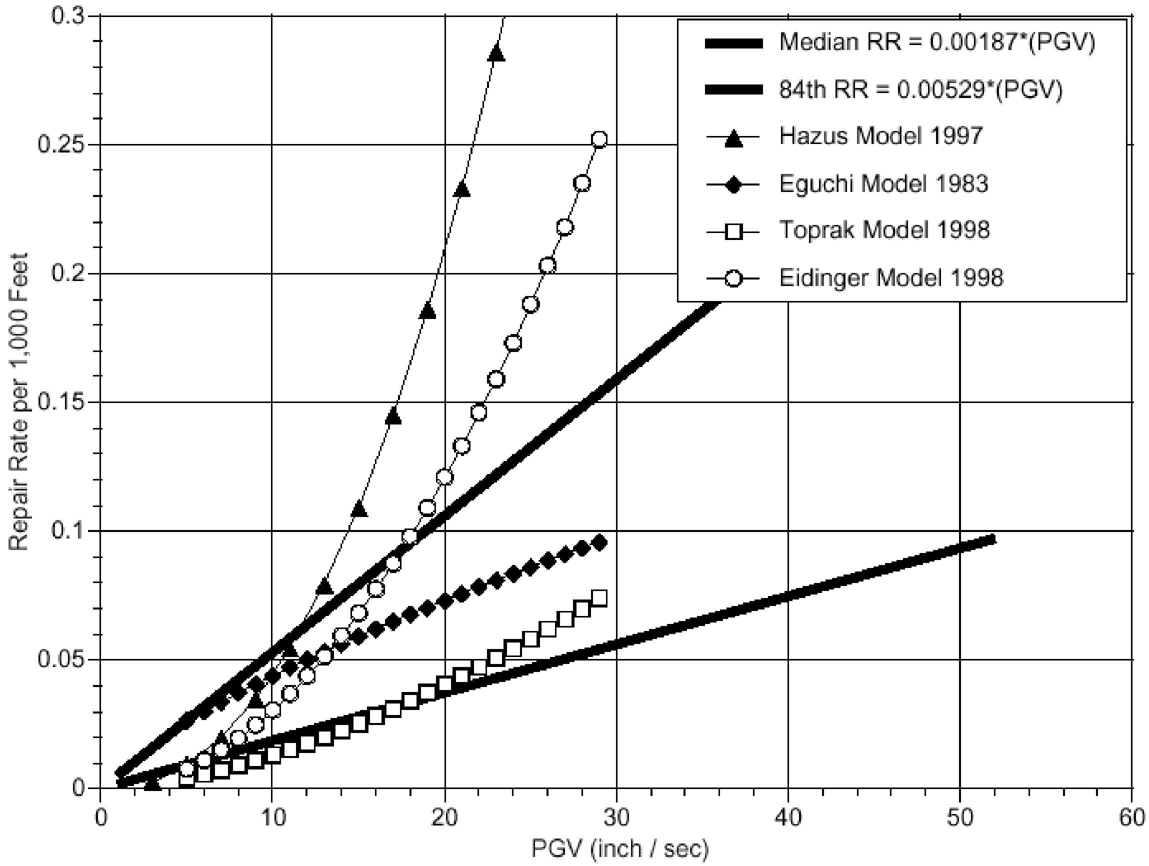


Figure 4-4. Comparison of Vulnerability Functions (Wave Propagation)

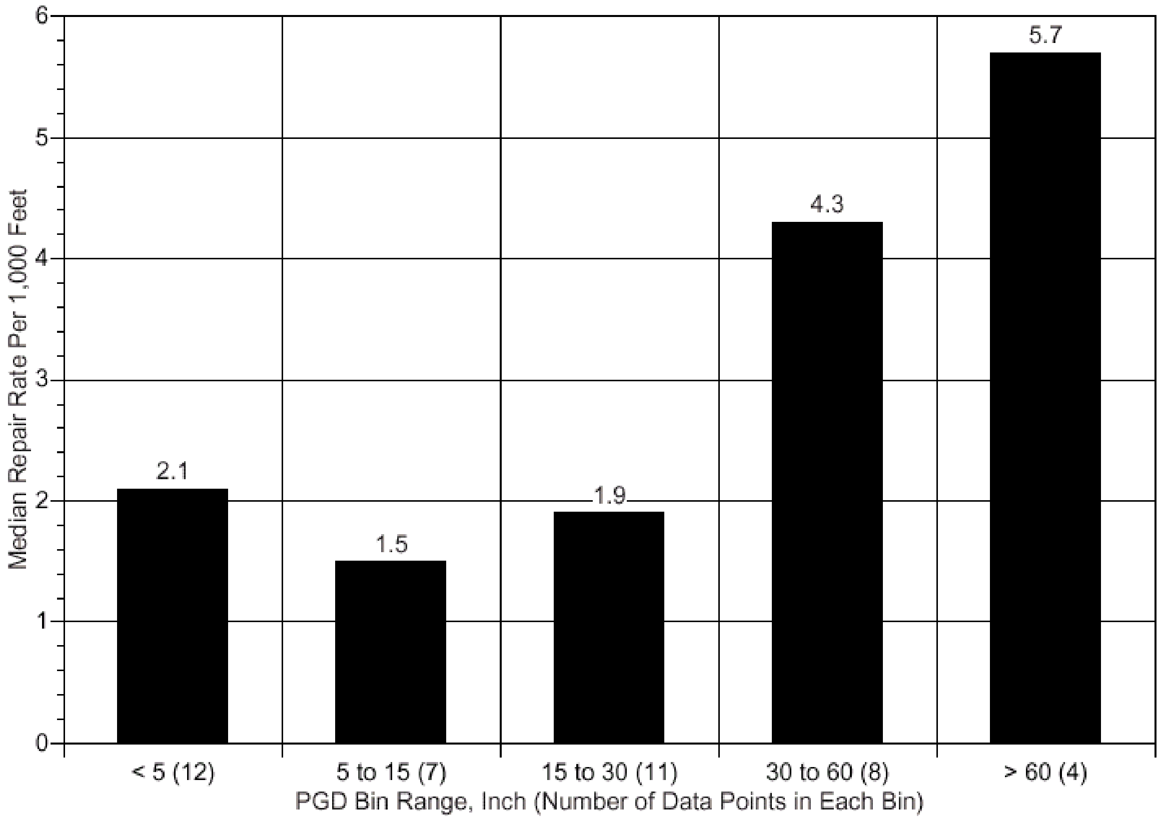


Figure 4-5. Bin Median Values (Permanent Ground Deformation)

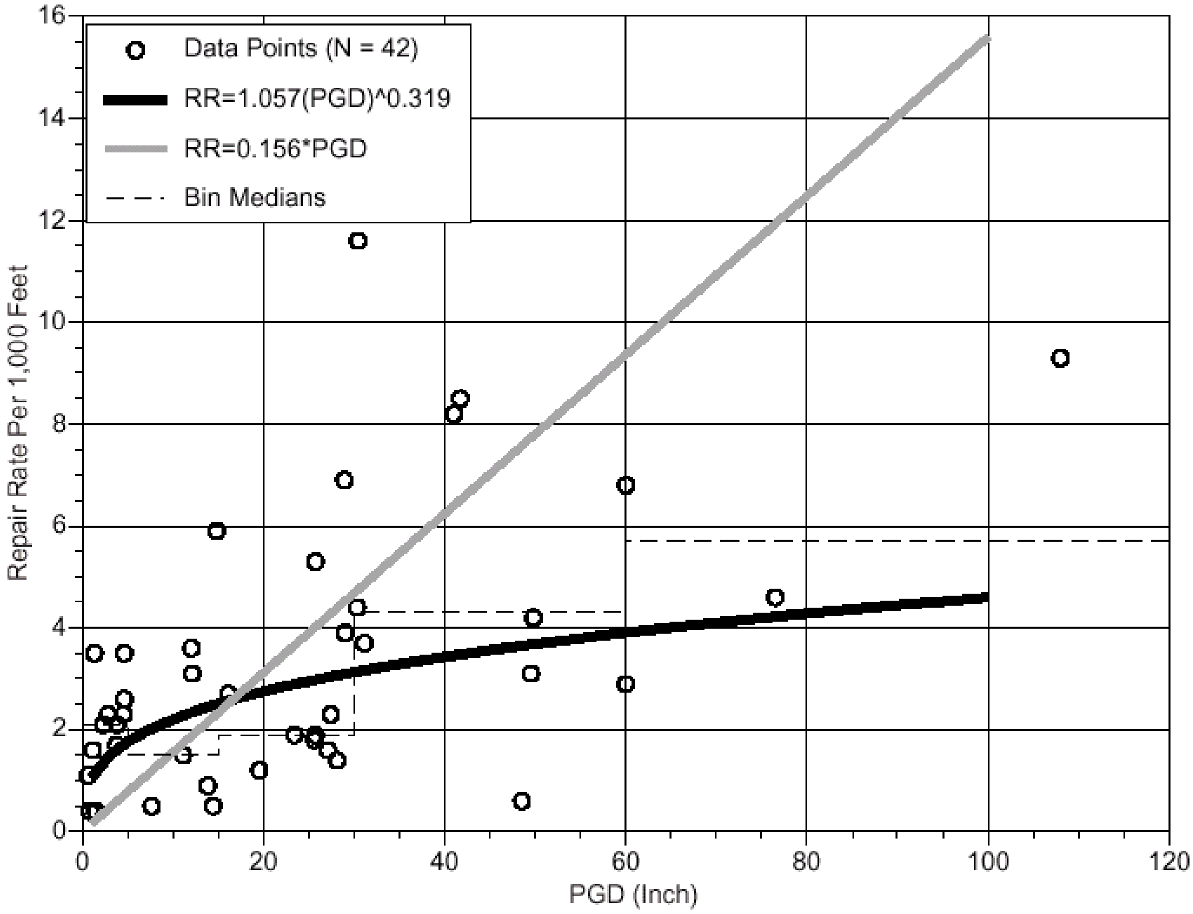


Figure 4-6. Vulnerability Functions (Permanent Ground Deformation)

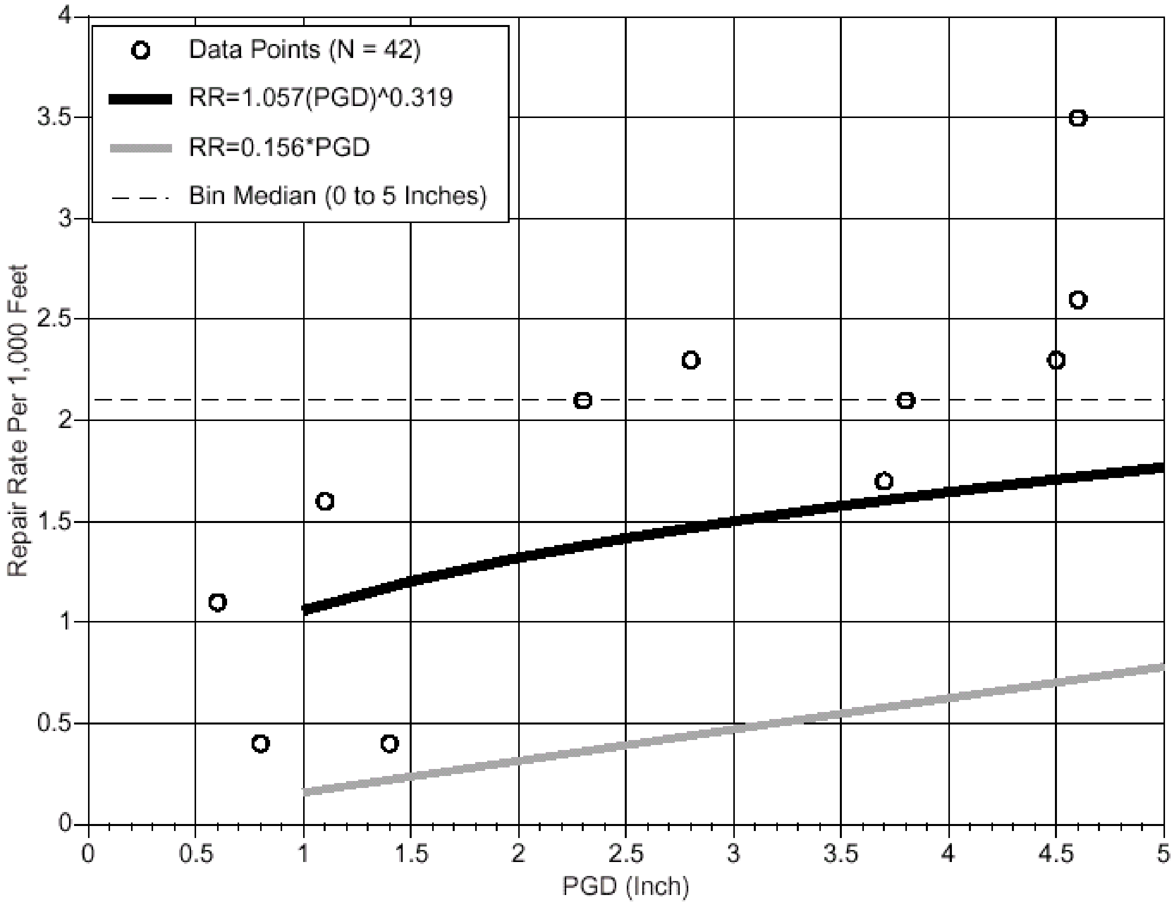


Figure 4-7. Vulnerability Functions (Permanent Ground Deformation) – Expanded Scale

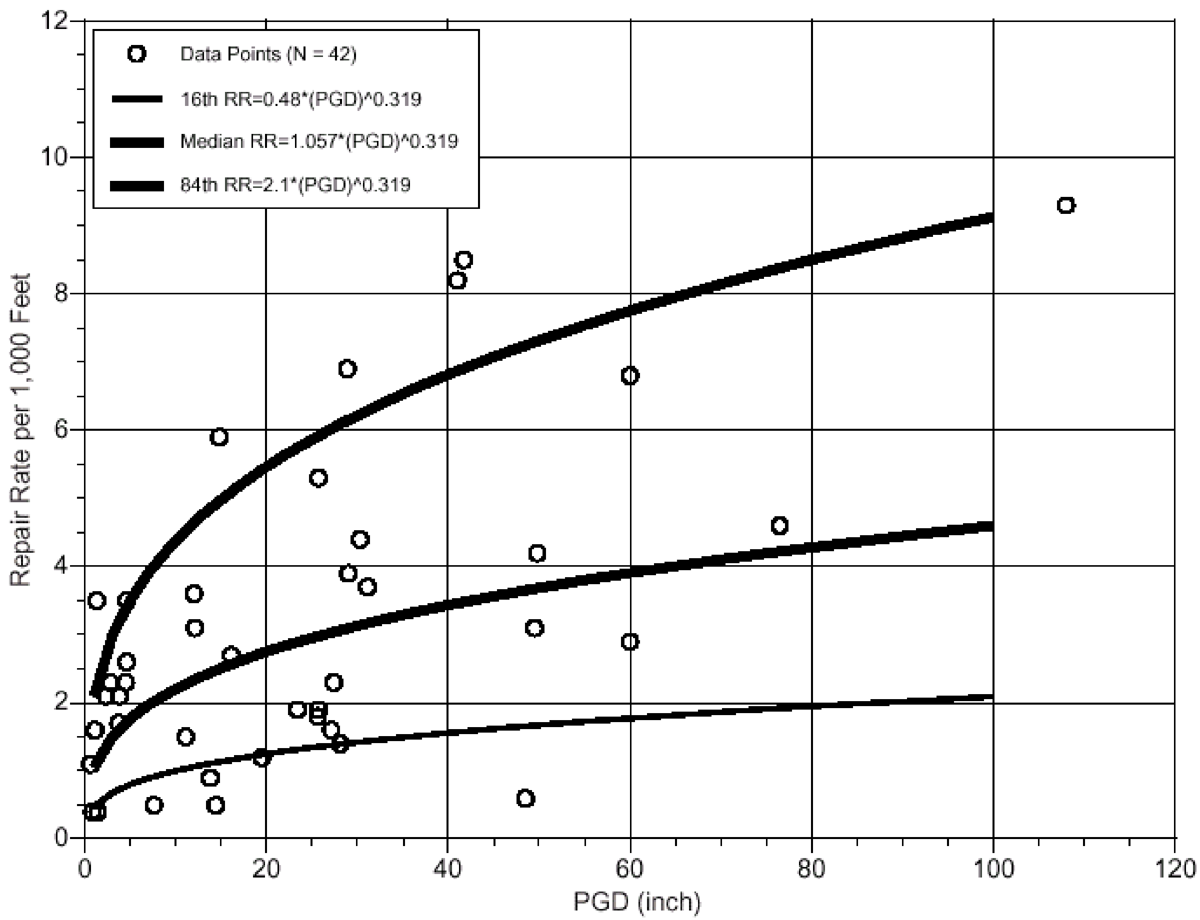


Figure 4-8. Median, 84th and 16th Percentile Functions (Permanent Ground Deformation)

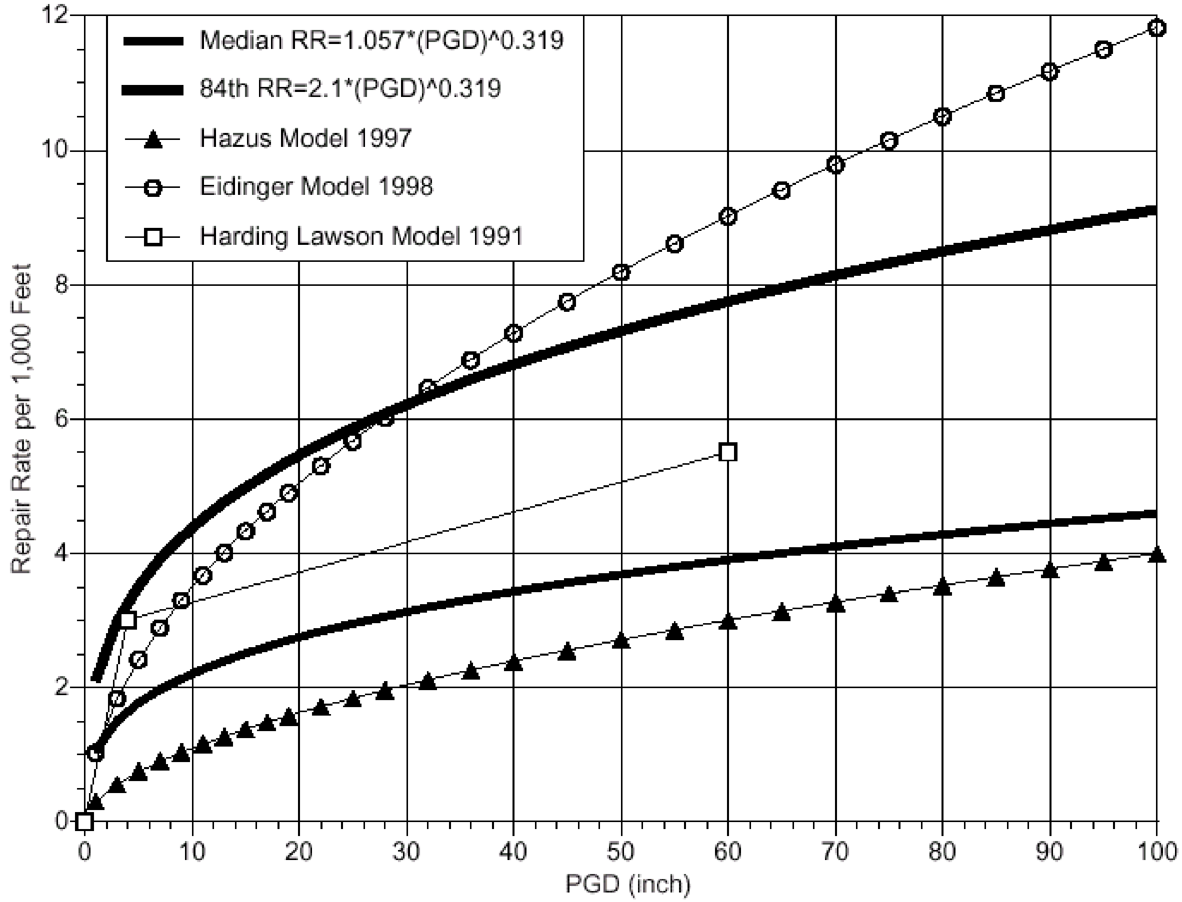


Figure 4-9. Comparison of Vulnerability Functions (Permanent Ground Deformation)

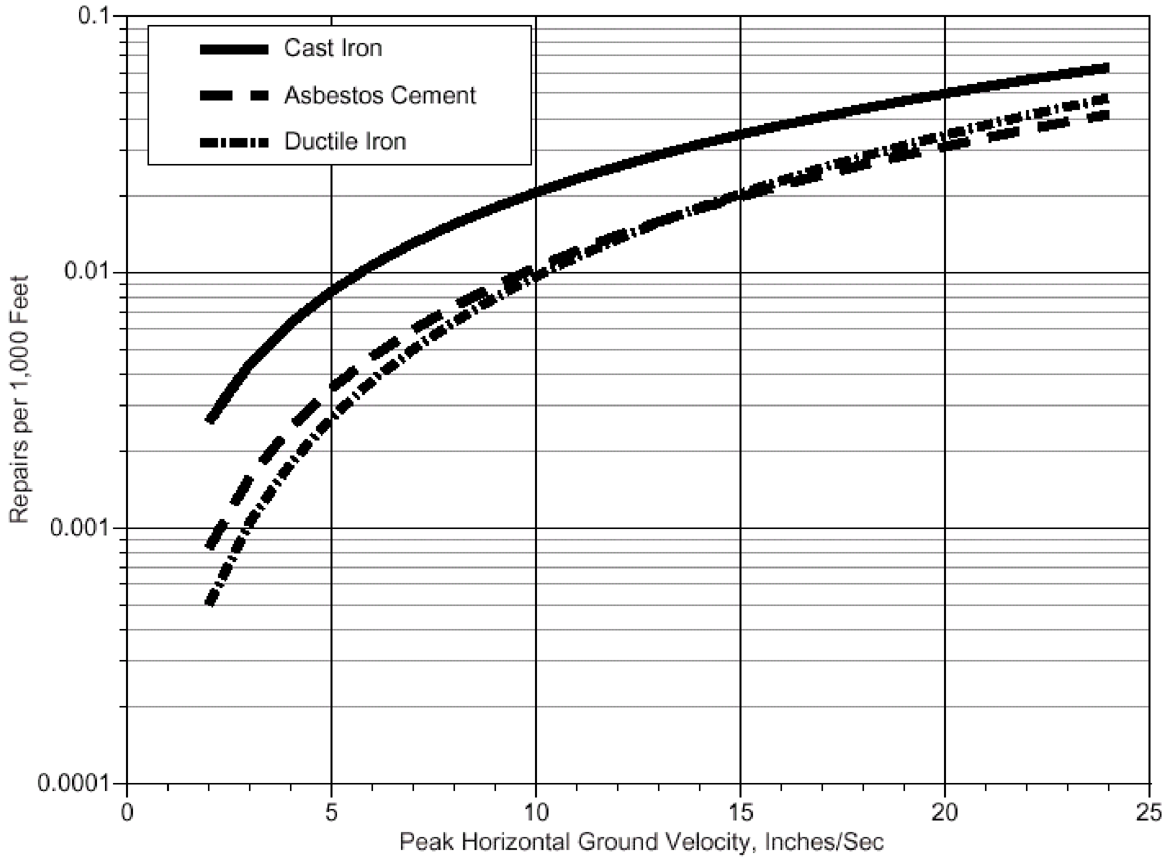


Figure 4-10. Pipe Damage – by Material – Regression using Data up to PGV=35 Inch/Sec

5.0 Water Tank Fragility Formulations

5.1 Factors that Cause Damage to Water Tanks

When applying fragility curves to water tanks, the analyst is often interested in several types of information such as the type and extent of damage, whether such damage impacts the functionality of the tank, the percent dollar loss to the tank, and the time needed to repair the damage to various states of operability. To assess this information, the following factors are needed:

- A description of the seismic hazard at the tank site. Depending on the type of fragility curve used, the hazard could be expressed in terms of peak ground acceleration (PGA) or a response spectrum at a particular damping level. If the tank site is projected to undergo some sort of liquefaction or landslide movement, then an estimate of the permanent ground deformation (PGD) that will affect the tank is needed. Tanks subject to fault offset are not covered by this report.

- A suite of fragility curves. Each curve will represent one damage state. For example, a damage state could be:

  - Anchor bolts stretched; tank remains functional.
  - Inlet-outlet pipe breaks, all water leaks out.
  - Bottom course buckles, weld fails, all water leaks out.
  - Roof system partially collapses into the tank.

- The replacement value of the tank. This represents the cost to build an identical volume tank at the same site. Often, the replacement value includes the value of demolition of the old tank. The value does not include the value of the land. The value should include all costs involved in replacing the tank, including planning, engineering, construction, construction management and inspection costs. A rough guideline to estimate these costs is provided in Appendix B.2. Appendix B.1 examines the relationship between damage states, repair cost and post-earthquake functionality.

- A correlation between the damage state and economic losses. For example, if the anchor bolt damage state occurs, then the direct repair cost for the tank is some percentage of the replacement value of the tank. Economic impacts other than direct damage can also occur, such as losses due to inundation of nearby locations, losses due to loss of water for fire fighting purposes, etc. It is beyond the scope of this effort to examine economic losses due to damage of tanks. See Eidinger and Avila [1999] for methods to treat all types of economic impacts of damage to various components of water systems.

Sections 5.1.1 through 5.1.8 describe failure modes that are known to have occurred to steel storage tanks. Implications about tank design are made where appropriate. Further details of these possible failure modes are documented in [NZNSEE 1986, Kennedy and Kassawara 1989].

5.1.1 Shell Buckling Mode

One of the most common forms of damage in steel tanks involves outward buckling of the bottom shell courses, a phenomenon termed “elephant foot.” Sometimes the buckling occurs over the full circumference of the tank. Buckling of the lower courses has occasionally resulted in the loss of tank contents due to weld or piping fracture and, in some cases, total collapse of the tank.

Tanks with very thin shells, such as stainless steel shells used for beer, wine and milk storage, have displayed another type of shell buckling mode involving diamond-shaped buckles a distance above the base of the tank.

5.1.2 Roof and Miscellaneous Steel Damage

A sloshing motion of the tank contents occurs during earthquake motion. The actual amplitude of motion at the tank circumference has been estimated, on the basis of scratch marks produced by floating roofs, to have exceeded several meters in some cases. For full or near full tanks, resistance of the roof to the free sloshing results in an upward pressure distribution on the roof. Common design codes [API, AWWA through the year 2000] do not provide guidance on the seismic design of tank roof systems for slosh impact forces. Modern tanks built after 1980 and designed to resist elephant foot buckling or other failure modes may still have inadequate designs for roof slosh impact forces.

In past earthquakes, damage has frequently occurred to the frangible joints between walls and cone roofs, with accompanying spillage of tank contents over the top of the wall. Extensive buckling of the upper courses of the shell walls has occurred. Floating roofs have also sustained extensive damage to support guides from the sloshing of contents. Steel roofs with curved knuckle joints appear to perform better, but these too have had supporting beams damaged from slosh impact forces.

Lateral movement and torsional rotations from ground shaking have caused broken guides, ladders and other appurtenances attached between the roof and the bottom plate. Lightweight wood roofs often used in water storage tanks are sometimes not designed for any seismic inertial loads and are especially vulnerable to sloshing-induced damage. Extensive damage to roofs can cause extensive damage to the upper course of a steel tank. However, roof damage or broken appurtenances, although expensive to repair, usually do not lead to more than a third of total fluid contents loss.

5.1.3 Anchorage Failure

Many steel tanks have hold-down bolts, straps or chairs. However, these anchors may be insufficient to withstand the total imposed load in large earthquake events and still can be damaged. The presence of anchors, as noted by field inspection, may not preclude anchorage failure or loss of contents.

Seismic overloads often result in anchor pull out, stretching or failure. However, failure of an anchor does not always lead to loss of tank contents.

5.1.4 Tank Support System Failure

Steel and concrete storage tanks supported above grade by columns or frames have failed because of the inadequacy of the support system under lateral seismic forces. This occurred to a steel/ cement silo in Alaska in 1964 and a concrete tank in Izmit, Turkey in 1999. Many elevated concrete water reservoirs failed or were severely damaged in the 1960 Chilean earthquake. Such failures most often lead to complete loss of contents.

5.1.5 Foundation Failure

Tank storage farms have frequently been sited in areas with poor foundation conditions. In past earthquakes like the one in Nigata in 1964, the liquefaction of materials under the tanks, coupled with imposed seismic moments on the tank base from lateral accelerations, resulted in base rotation and gross settlements on the order of several meters.

In other cases on firm foundations, fracture of the baseplate welds occurred in tanks not restrained or inadequately restrained against uplift. In these cases, seismic accelerations resulted in uplift displacements on the tension side of the tank, up to 14 inches recorded in the 1971 San Fernando earthquake. Since the baseplate is held down by hydrostatic pressure of the tank contents, the base weld is subject to high stresses and fracture may result. In some cases, the resulting loss of liquid has resulted in scouring the foundation materials in the vicinity, reducing support to the tank in the damaged area and exacerbating the damage.

A large underground reinforced concrete reservoir, part of the Balboa water treatment plant, suffered severe damage in the 1971 San Fernando earthquake, apparently a consequence of foundation failure. The walls, roof slab, floor slab and some columns of this 450-foot by 450-foot by 40-foot-high reservoir were extensively damaged, particularly along construction joints. Damage was apparently caused by movement of the filled ground, which had about 50% relative density due to consolidation up to 1.5 feet, and sliding produced by ground shaking.

Another common cause of failure is severe distortion of the tank bottom at or near the tank side wall due to a soil failure such as soil liquefaction, slope instability, or excessive differential settlement. Soil failures are best prevented through proper soil compaction prior to placement of the tank and through the use of a reinforced mat foundation under the tank.

Another less common cause of failure results from tank sliding. There is no known case where an anchored tank with greater than a 30-foot diameter has slid. Sliding is possible a concern for unanchored smaller diameter tanks.

5.1.6 Hydrodynamic Pressure Failure

Tensile hoop stresses can increase due to shaking-induced pressures between the fluid and the tank, and can lead to splitting and leakage. This phenomenon has occurred in riveted tanks where leakage at the riveted joints resulted from seismic pressure-induced yielding. This happens more often in the upper courses. No known welded steel tank has actually ruptured because of seismically induced hoop strains; however, large tensile hoop stresses can contribute to the likelihood of elephant foot buckling near the tank base due to overturning moment.

Hydrostatic pressure failure may also be a cause for failure in concrete tanks, due to excessive hoop tensile forces in the steel reinforcement. This was the apparent mode of damage for a concrete tank near Palo Alto in the 1989 Loma Prieta earthquake. This failure mode may be aggravated by corrosion of the hoop direction prestressing wires.

5.1.7 Connecting Pipe Failure

One of the more common causes of loss of tank contents in earthquakes has been the fracture of piping at connections to the tank. This generally results from large vertical displacements of the tank caused by tank buckling, wall uplift or foundation failure. This happened to steel tanks in the 1992 Landers earthquake. Failure of rigid piping that connects to adjacent tanks has also been caused by relative horizontal displacements of the tanks. Piping failure has also resulted in extensive scour in the foundation materials.

Another failure mode has been the breaking of pipe that enters the tank from underground, due to the relative movement of the tank and the pipe. This occurred several times during the 1985 Chilean earthquake.

Kennedy and Kassawara have suggested that almost any type of flexibility loop in a pipe between the tank and the independent piping supports should be sufficient for a low probability of failure at PGA levels up to 0.5g. However, if there is a straight run (i.e., no flexibility loop) from the point where the pipe is independently rigidly supported, and there are relative anchor motions, Kennedy and Kassawara suggest checking the tank nozzle and tank shell. For example, rigid overflow pipes attached to steel tanks have exerted large forces on the tank wall supports due to the relative movement of the tank to the ground. The wall supports of one such pipe tore out of the shell of an oil tank in Richmond in the 1989 Loma Prieta earthquake; the pipe support failure left a small hole in the shell around mid-height of the tank.

5.1.8 Manhole Failure

Loss of contents has occurred because of overloads on manhole covers. Manhole failure has occurred in thin-walled, stainless steel tanks used for wine storage. It has also occurred at manhole cover doubler plates when these plates extend low enough in the bottom course to be highly strained in the event of elephant foot buckling.

5.2 Empirical Tank Data Set

To examine the empirical performance of tanks, this report updates and supplements the available empirical data sets described in Appendix B. The procedure was as follows:

- The inventory of 424 tanks developed by Cooper [Cooper, 1997] was reviewed from source material and, for the most part, was found to be correct. In a few instances, the damage states for broken pipes were adjusted as follows: if damage to a pipe created only slight leaks on minor repairs such as damage to an overflow pipe, the damage state was assigned equal to 2 (same as O'Rourke and So). However, if damage to a pipe led to complete loss of contents or a complete breaking of the inlet-outlet line, then the damage state was assigned equal to 4. This is more consistent with the performance of the tank. A broken inlet-outlet line puts the tank out of service at DS=4, while a leaking overflow line does not put the tank out of service at DS=2. A damaged inlet/outlet line usually means a substantial uplift of the base of the tank has occurred. Substantial buckling in the upper courses was defined as DS=2 by O’Rourke and So, but is defined as DS = 3 in the current effort. This reflects that wall buckling has occurred, without leakage of tank contents, and that this type of damage is more costly to repair than damage to the roof system alone. Because of incomplete descriptions of the actual damage to some tanks, the definition of damage state between DS = 2, DS = 3 and DS = 4 is sometimes left to judgment.

- The ground motion parameters for the 1964 Alaskan earthquake were established based on conversion from MMI to PGA, and by examining attenuation models for subduction zone earthquakes. In this way, the significant set of damaged tanks from that earthquake—32 out of 39 tanks—can be added into the fragility analysis. It should be understood that no accelerometer recordings are available for this earthquake, MMI maps are not all that precise, and the 90-second to 180-second duration of strong shaking from this event greatly exceeds the duration of shaking from most other events.

- The ground motion parameters for the 1983 Coalinga earthquake are refined using a combination of attenuation relationships, recorded instrumental motions and information from Hashimoto [1989]. O'Rourke and So approximated all tanks to have experienced 0.71g. The only near-field instruments were located at the Pleasant Valley pump station, where the horizontal recorded motions were 0.54g and 0.45g recorded by the instrument in switchyard, and 0.28g and 0.33 g recorded by the instrument in the basement of a building. These instruments were located 9 km from the epicenter, which, using attenuation for rock site, gives median PGA = 0.37g. With regards to the MMI scale, the highest intensity suggested for this earthquake is MMI VIII, which roughly translates to a PGA = 0.26g to 0.45g according to the McCann relationship. The resulting ground motions for the bulk of the oil tanks in the area are from 0.39g to 0.62g, which is lower than the 0.71g assumed by O'Rourke and So. While some tanks may have experienced the very high g levels of 0.71g, it is also likely that some tanks experienced more moderate values under 0.4g.
- 13 tanks from the 1984 Morgan Hill earthquake were added to the analysis.
- 38 tanks from the 1991 Costa Rican earthquake were added to the analysis.
- 7 tanks from the 1971San Fernando earthquake were added to the database.
- 3 tanks from the 1983 Coalinga earthquake were added to the database.
- 5 tanks from the 1985 Chilean earthquake were added to the database.
- 3 tanks from the 1986 Adak, Alaska earthquake were added to the database.
- 3 tanks from the 1987 Whittier earthquake were added to the database.
- 11 tanks from the 1987 New Zealand earthquake were added to the database.
- Individual tanks from the 1975 Ferndale, 1980 Ferndale, 1980 Greenville, 1972 Managua and 1978 Miyagi-ken-ogi earthquakes were added to the database.

An additional 1,670 tanks were exposed to relatively low levels of 0.03 to about 0.10g of ground motions in the 1989 Loma Prieta earthquake. Only two of these tanks are known to have suffered slight damage to roof structures. For purposes of developing fragility curves, this large population of tanks is considered to be strong evidence of the likelihood that tanks at ground motions at 0.10g or below do not suffer damage.

All told, 532 tanks in the database experienced strong ground motions of 0.10g or higher. An additional 1,670 tanks in the database experienced lower level ground motions of 0.03g to 0.10g. The analysis that follows uses only the 532 tanks with ground motions of 0.10g or higher.

Table 5-1 summarizes the empirical database. Tables B-8 through B-18 provide the complete tank database.

Event	No. of Tanks	PGA Range (g)	Average PGA (g)	PGA Source
1933 Long Beach	49		0.17	Cooper 1997
1952 Kern County	24		0.19	Cooper 1997
1964 Alaska	39	0.20 to 0.30	0.22	This report
1971 San Fernando	27	0.20 to 1.20	0.51	Wald et al 1998
1972 Managua	1	0.50	0.50	Hashimoto 1989
1975 Ferndale	1	0.30	0.30	Hashimoto 1989
1978 Miyagi-ken-ogi	1	0.28	0.28	Hashimoto 1989
1979 Imperial Valley	24	0.24 to 0.49	0.24	Haroun 1983
1980 Ferndale	1	0.25	0.25	Hashimoto 1989
1980 Greenville	1	0.25	0.25	Hashimoto 1989
1983 Coalinga	48	0.20 to 0.62	0.49	This report, Hashimoto 1989
1984 Morgan Hill	12	0.25 to 0.50	0.30	This report
1985 Chile	5	0.25	0.25	Hashimoto 1989
1986 Adak	3	0.20	0.20	Hashimoto 1989
1987 New Zealand	11	0.30 to 0.50	0.42	Hashimoto 1989
1987 Whittier	3	0.17	0.17	Hashimoto 1989
1989 Loma Prieta	141	0.11 to 0.54	0.16	Cooper 1997
1989 Loma Prieta (Low g)	1,670	0.03 to 0.10	0.06	This report
1991 Costa Rica	38	0.35	0.35	This report
1992 Landers	33	0.10 to 0.56	0.30	Cooper 1997, Ballantyne and Crouse 1997, Wald et al 1998
1994 Northridge	70	0.30 to 1.00	0.63	Brown et al 1995, Wald et al 1998
Total (excl. low g)	**532**	**0.10 to 1.20**	**0.32**	

Table 5-1. Earthquake Characteristics for Tank Database

Table 5-2 provides a breakdown of the number of tanks in various damage states. The value in the PGA column in Table 5-2 is calculated as the average PGA for all tanks in a PGA range; the ranges were set in steps of 0.10g. (Note: one tank was in damage state 5 and collapsed because of collapse of an adjacent tank; it was removed from the database used for developing fragilities).

PGA (g)	All Tanks	DS = 1	DS = 2	DS = 3	DS = 4	DS = 5
0.10	4	4	0	0	0	0
0.16	263	196	42	13	8	4
0.26	62	31	17	10	4	0
0.36	53	22	19	8	3	1
0.47	47	32	11	3	1	0
0.56	53	26	15	7	3	2
0.67	25	9	5	5	3	3
0.87	14	10	0	1	3	0
1.18	10	1	3	0	0	6
Total	**532**	**331**	**112**	**47**	**25**	**16** [1]

Note 1. Most of the collapsed tanks were made of riveted steel. Application of Damage State 5 for welded steel tanks should be used with caution.

Table 5-2. Complete Tank Database

5.2.1 Effect of Fill Level

Fragility curves were calculated for a variety of fill levels in the tank database, as shown in Table 5-3. 'A' represents the median PGA value (in g) to reach or exceed a particular damage state, and Beta is the lognormal standard deviation. 'N' is the number of tanks in the particular analysis.

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

Table 5-3. Fragility Curves, Tanks, as a Function of Fill Level

The following trends can be seen in Table 5-3:

- Tanks with low fill levels (below 50%) have much higher median acceleration levels to reach a particular damage state than do tanks that are at least 50% filled.
- Tanks with low fill levels are not known to experience damage states 4 or 5 and showing elephant foot buckling with leakage or other damage leading to rapid loss of all contents or collapse. Thus, no values are given.
- Tanks with fill levels of 90% or higher have moderately lower fragility levels than tanks with fill levels of 50% or higher. Most water system distribution tanks are kept at fill levels between 80% and 100%, depending upon the time of day. If no other attributes of a given water storage tank are known, then the fragilities for the 90% fill levels or higher should be used. Oil tanks can often have fill levels of less than 50%.
- The Beta values are mostly = 0.80. This reflects the large uncertainty involved in the tank database. For example, site PGA values were generally estimated using attenuation or MMI-to-PGA conversions (e.g., average horizontal motion), but in some cases, the PGA reported is based on the largest of two horizontal PGA components from a nearby

accelerometer. Site soil conditions are undifferentiated and could have been rock or soil, which has a significant impact on spectral accelerations for both the impulsive and convective modes of liquid motions in a tank. Tank construction attributes like wall thickness were not considered and damage descriptors were not always precise. Beta values would normally be in the 0.30 to 0.45 range for a tank-specific calculation; however, the regression analysis showed a larger standard error in the curve-fitting process for most cases with Beta under 0.80.

- The fragility values for the DS=5 or collapse show little variation and small beta values. This reflects that only a small number of tanks actually collapsed due to gross movement of the shell. The collapse mechanisms could have been initiated by gross elephant foot buckling or gross roof damage, possibly caused by upper level diamond buckling. Possibly, a better way to describe this damage state is to assume that about 6% of all tanks reaching damage state 2 or above actually collapse, since 16 of 200 tanks with some form of damage collapsed. Most of the collapsed tanks were riveted steel, and this attribute is not common in most steel tanks built after 1950, so this damage state should be used with caution.

The empirical fragility parameters in Table 5-3 can be compared to those suggested by O'Rourke and So in Table B-7. The following observations are made:

- The empirical fragility parameters for Fill •50% in Table 5-3 is based on a sample size of N=251 tanks. The empirical fragility curves (O’Rourke and So) for Fill •50% in Table B-7 is based on a sample size of N=133 tanks. The largest difference between the two analyses is for DS=2, where the complete dataset has a median A = 0.18g, and the O’Rourke and So dataset has a median A = 0.49g. Table 5-4 provides the raw data used to prepare the results in Table 5-3, and it is clear that the majority of tanks with fills • 50% have sustained some type of damage at PGA=0.18g or above. One reason for this large difference is that the O’Rourke and So analysis excluded all damage from the Alaskan earthquake in which 32 of 39 tanks were damaged.

PGA (g)	All Tanks	DS = 1	DS = 2	DS = 3	DS = 4	DS = 5
0.10	1	1	0	0	0	0
0.17	77	22	32	12	8	3
0.27	43	16	13	10	4	0
0.37	22	3	11	4	3	1
0.48	25	12	9	3	1	0
0.57	48	22	14	7	3	2
0.66	15	4	2	3	3	3
0.86	10	7	0	0	3	0
1.18	10	1	3	0	0	5
Total	**251**	**88**	**84**	**39**	**25**	**15**

Table 5-4. Tank Database, Fill ≥ 50%

5.2.2 Effect of Anchorage

Two sets of HAZUS fragility curves are presented in Table B-7 [HAZUS, 1997]. These curves are based on analytical development of fragility curves for anchored and unanchored steel tanks that had been designed to various editions of the AWWA D100 standard from 1950 to 1990. The HAZUS curves suggest that anchored tanks should perform better than unanchored tanks.

The most recent 1996 edition of the AWWA D100 standard has made a significant change to the design compressive allowable for anchored tanks, as compared to prior editions of that standard. The AWWA D100-1996 allows the compressive allowable for unanchored tanks to take credit for the beneficial effects of internal pressure, a condition that is not allowed for anchored tanks. It is unclear as to the rationale for this unequal treatment, and it would be expected that tank owners may tend towards unanchored tank design to achieve cost savings while implicitly accepting worse tank performance in future earthquakes. Tanks designed to the AWWA D100-1996 standard are expected to be even more resistant to elephant foot buckling failure modes than unanchored tanks designed to the AWWA D100-96.

The empirical database was analyzed to assess the relative performance of anchored versus unanchored tanks. All tanks in the empirical database were designed prior to the AWWA D100-1996 code, and likely used equal compressive stress allowable for the tank shell, whether anchored or unanchored. Since fill level has been shown to be very important in predicting tank performance, only tanks with fill levels • 50% are considered in this analysis. Table 5-5 shows the empirical database for anchored tanks with fill levels •50%. Table 5-6 shows the empirical database for unanchored tanks with fill levels • 50%. Tanks in Table 5-4 with uncertain anchorage were assumed to be unanchored.

PGA (g)	All Tanks	DS = 1	DS = 2	DS = 3	DS = 4	DS = 5
0.10	0	0	0	0	0	0
0.19	13	13	0	0	0	0
0.27	16	14	1	1	0	0
0.39	5	3	1	0	1	0
0.50	7	6	1	0	0	0
0.58	5	2	1	1	0	1
0.90	1	1	0	0	0	0
1.20	1	0	1	0	0	0
Total	**46**	**37**	**5**	**2**	**1**	**1**

Table 5-5. Anchored Tank Database, Fill ≥ 50%

PGA (g)	All Tanks	DS = 1	DS = 2	DS = 3	DS = 4	DS = 5
0.10	1	1	0	0	0	0
0.17	65	10	32	12	8	3
0.27	27	2	12	9	4	0
0.36	17	0	10	4	2	1
0.47	19	7	8	3	1	0
0.56	43	20	13	6	3	1
0.66	15	4	2	3	3	3
0.86	9	6	0	0	3	0
1.18	9	1	2	0	0	6
Total	**205**	**51**	**79**	**37**	**24**	**14**

Table 5-6. Unanchored Tank Database, Fill ≥ 50%

As seen in Table 5-7, the empirical evidence for the benefits of anchored tanks is clear. The median PGA value to reach various damage states is about 3 to 4 times higher for anchored tanks than for unanchored tanks. It should be noted, however, that the anchored tank database (N=46) is much smaller than the unanchored tank database (N=251), and fill levels may not have been known for all tanks in the anchored tank database. Also, it has been suggested by the SQUG steering group [personal communication, A. Schiff, 2000], that the anchored tank database by Hashimoto [1989] may include PGA values that may have been higher than actually experienced by some tanks. Also, some of the anchored tanks in the database are relatively smaller—under 100,000 gallon capacity—than most other tanks in the database. Even with these considerations, the empirical evidence strongly suggests that anchored tanks outperform unanchored tanks.

DS	A, g	Beta	A, g	Beta	A, g	Beta	A, g	Beta	A, g	Beta
DS≥2	0.18	0.80	0.71	0.80	0.15	0.12	0.30	0.60	0.15	0.70
DS≥3	0.73	0.80	2.36	0.80	0.62	0.80	0.70	0.60	0.35	0.75
DS≥4	1.14	0.80	3.72	0.80	1.06	0.80	1.25	0.65	0.68	0.75
DS=5	1.16	0.80	4.26	0.80	1.13	0.10	1.60	0.60	0.95	0.70
	Fill ≥ 50% All N=251		Fill ≥ 50% Anchored N=46		Fill ≥ 50% Unanchored N=205[1]		Near Full Anchored HAZUS		Near Full Unanchored HAZUS	

Note 1. The low beta values (0.12, 0.10) reflect the sample set. However, beta = 0.80 is recommended for use for all damage states for regional loss estimates for unanchored steel tanks with fill ≥ 50% unless otherwise justified.

Table 5-7. Fragility Curves, Tanks, As a Function of Fill Level and Anchorage

When comparing the current empirical fragility curves to the HAZUS curves, the following observations can be made:

- The HAZUS curves for unanchored tanks are in the same range as the empirical curves. Note that the empirical curves in Table 5-7 are for tanks with fill • 50%, while the HAZUS curves are for nearly full tanks. Table 5-3 shows a modest decrease in seismic performance as fill level goes up.
- The HAZUS curves indicate a marked increase in capacity for anchored tanks as compared to unanchored tanks, and the empirical database shows an even larger increase. As the empirical database for anchored tanks is small for DS=3 and higher, the very high PGA values suggested (2.36 to 4.26g for DS=3 to 5) are based on limited extrapolation and possibly should not be used directly. Instead, temperance between the empirical database and the HAZUS values for anchored tanks might be appropriate for simple loss estimation studies.

5.2.3 Effect of Permanent Ground Deformations

Insufficient information exists in the empirical data set to establish fragility curves for tanks subjected to PGDs from landslides or liquefaction. The following fragility levels are based on judgment and are incorporated into the fragility parameters in Tables 5-8 through 5-16:

- For steel tanks, a 50% change of substantial tank damage would occur if a steel tank experiences a differential offset of 36 inches. Differential offset means the amount of PGD varies from one end of the tank to the other end by 36 inches. This damage state corresponds to a complete loss of the tank.
- For concrete tanks, the amount of PGD needed to reach a similar damage state is assumed to be 24 inches. This reflects the assumed lower tolerance for concrete tanks to sustain

differential settlements or movements as compared to that of steel tanks. This damage state corresponds to a complete loss of the tank.

- For open cut reservoirs, the amount of PGD needed to cause widespread damage to the roof structure is assumed to be 8 inches. This report does not provide fragilities for failure of embankment dams.
- Damage to pipes attached to the tank due to PGDs would normally be captured in the analysis of the pipelines. Offsets of a few inches would likely damage attached pipes that do not have the capability to absorb any significant displacements. Although this damage would put the tank out of service, it is relatively inexpensive to repair.

5.3 Analytical Fragility Curves

Section 5.2 provides fragility curves based on the empirical performance of at-grade steel tanks in prior earthquakes. These fragility curves may be appropriate for simplified loss estimation for large numbers of tanks. However, use of these empirical fragility curves to estimate the actual performance of a specific tank may lead to inappropriate conclusions. In part, this is because the attributes of a specific tank may not match the "average" attributes of the many tanks in the empirical database.

The fragility curves for a specific tank can be derived from analysis. The general analytical approach to developing tank-specific fragility curves is as follows:

1. Perform a deterministic evaluation of the tank being considered. This procedure follows the normal building codes and standards used in design (AWWA D100, API 650, etc.), with the general exception that no energy absorption 'Rw' factor is allowed (i.e., use $R_w = 1$). This evaluation will yield a number of possible damage states for the tank, such as:
   - Failure of the weld at the bottom of a steel tank
   - Yielding of the steel in hoop tension
   - Failure of the anchor bolts holding down the tank
   - Sliding of the tank
   - Breakage of the inlet-outlet pipe

For each of these damage states, the deterministic analysis will provide a frequency and a spectral acceleration needed to get to the code-defined allowable stress limit:

f_{ds} = fundamental frequency of the tank for the loading that leads to this damage state

A_{ds} = code-based spectral acceleration needed to reach this damage state

2. It is usual that most building codes and standards imply a safety factor between the code design level and the actual level of shaking needed on average to cause the damage state. This median spectral acceleration, $\hat{A}_{ds}$, can be considered related to the code-based spectral acceleration as follows:

$\hat{A}_{ds} = F * A_{ds}$ [eq 5-1]

F = factor of safety

For example, if a concrete shear wall is determined by code-based analysis to have a capacity A_{ds} , then $\hat{A}_{ds}$ can be determined by increasing A_{ds} by a factor f_1, because the code formula is a conservative approximation of test results; by a factor f_2 because actual concrete strengths usually exceed minimum strengths specified in the design documents; and by a factor f_3 because the detailing will result in a wall ductility μ.

3. The capacity distribution for the various damage states can be described with a lognormal distribution, β. The value for β can be determined from past tests. Using the example in step 2 above, scatter from past test data can be derived for concrete shear strength, compressive strength and ductility, or β_1, β_2, and β_3. The total β is then the square root of the sum of the squares of the individual βs. Randomness and uncertainty from other factors, particularly the ground motions, can be added in a similar fashion.

In general, the total factor of safety F is composed of:

- A strength factor based on the variability in material strengths and workmanship.
- An inelastic energy absorption factor related to the particular damage state, the behavior of the materials involved and the overall ductility of the structure.
- Damping used in the code-based analysis (often 5%) versus that actually expected associated with the damage state (often higher than 5%, but sometimes lower for certain tank-specific damage states).
- Modeling assumptions (e.g., equivalent elastic static lateral force method versus inelastic dynamic time history). Simplified modeling assumptions usually lead to conservative predictions of load, but introduce some uncertainty.
- Model combination methods (e.g., single degree of freedom system versus combined multi-modal response). Often, attributing the entire mass of a structure into the fundamental mode will overpredict internal forces in the structure.
- Soil-structure interaction and wave incoherence effects.

The total F is a product of the above individual factors. Not all these factors affect every damage state for every structure.

The analytically derived damage algorithms described in Section 5.4 are based on an assumed duration of strong ground shaking of around 15 to 20 seconds. This range is typical for tanks on rock or firm soil sites subjected to crustal earthquakes of moderate magnitude; M 6 to M 7.5 is typical for California. Damage states that are sensitive to repeated cyclic responses can occur at lower accelerations in longer duration earthquakes. Some method to quantitatively include duration into loss estimates should be introduced into loss estimation efforts for low magnitude earthquakes of M 6 or below, or very high magnitude earthquakes of M 7.6 or above.

5.4 Representative Fragility Curves

Representative fragility curves for 11 types of water distribution system tanks are described in this section. All tanks are assumed to have height-to-diameter (H/D) ratios under 0.75. The larger volume water tanks with more than 2 million gallons of capacity will usually have lower H/D ratios.

The fragility curves in Tables 5-8 through 5-16 are based on the average results of analytical calculations for a variety of tanks and are supplemented by engineering judgment. This report does not provide detailed strength-of-mechanics calculations that were used to prepare these fragility curves; however, Appendix B.7 contains a sample calculation to compute the onset of elephant foot buckling for a typical anchored steel tank. These curves should *only* be considered representative of fragilities for specific tanks and should always be adjusted for tank-specific conditions.

Bandpadhyay et al [1993] and Kennedy and Kassawara [1989] give detailed methods for analyzing tanks and calculating tank-specific fragilities.

For specific tanks, developing tank-specific fragility curves is recommended. These will take into account tank-specific features, such as height, diameter, wall thickness, strength of materials, fill height, available freeboard, methods to attach pipes, type of foundation, type of roof, density of liquid—which is important for oil products, local soil conditions, etc. Many combinations of these parameters are possible.

The following fragility curves use response spectral ordinates of 5% damped horizontal response spectra at a particular impulsive mode frequency. This is considered a better indicator of seismic forces than PGA. Sloshing mode failure modes should be based on the response spectral ordinates of 0.5% damped horizontal response spectra at a particular convective mode frequency. Fragilities with both impulsive and convective modes, like overturning moment, are based on the impulsive mode frequency and spectral ordinate and can assume a ratio of convective mode to impulsive mode response spectral values for preliminary analyses.

Each representative fragility curve provides the median spectral acceleration and the beta, or lognormal standard deviation, that represents uncertainty only. Randomness in ground motions is not included in Tables 5-8 through 5-18 and must be accounted for in specific analyses. If a single beta is desired to represent both uncertainty and randomness, then the beta values in Tables 5-8 through 5-18 can be converted to a total beta as follows:

$$\beta_{total} = \sqrt{\beta_u^2 + \beta_r^2} \qquad \text{[eq. 5-2]}$$

where

β_r is typically around 0.40 for high-magnitude crustal earthquakes in California, and perhaps as high as 0.60 for earthquakes affecting the Eastern United States. It is beyond the scope of this report to specify β_r in detail. Tables 5-8 through 5-18 contain data for the following configurations:

- Unanchored redwood tank (50,000 - 500,000 gallons)
- Unanchored post-tensioned circular concrete tank (1,000,000+ gallons)

- Unanchored steel tank with integral shell roof (100,000 - 2,000,000 gallons)
- Unanchored steel tank with wood roof (100,000 - 2,000,000 gallons)
- Anchored steel tank with integral steel roof (100,000 - 2,000,000 gallons)
- Unanchored steel tank with integral steel roof (2,000,000+ gallons)
- Anchored steel tank with wood roof (2,000,000+ gallons)
- Anchored reinforced (or prestressed) concrete tank (50,000 - 1,000,000 gallons)
- Elevated steel tank with no seismic design
- Elevated steel tank with nominal seismic design
- Roof over open cut reservoir

The fragility curves in Tables 5-8 through 5-18 assume that the tank is full (filled to the overflow level) at the time of the earthquake. The following paragraphs describe the basis and intended usage of these fragility curves.

Wood Tanks at Grade. Use Table 5-8.

Hazard	Damage State	Damage Factor	Ground Shaking Median A (g)	Ground Shaking Beta	Ground Shaking Freq. (Hz)	Ground Failure Median PGD (inch)	Ground Failure Beta	Functionality Code	Logic Code
Ground Shaking	Tank slides breaks inlet line.	0.50	1.00	0.50	7			0	1
Ground Shaking	Wall-to-floor connection fails due to uplift.	0.15	2.25	0.50	7			0	2
Ground Shaking	Bars stretch, tank leaks.	0.15	2.25	0.50	7			1	3
Ground Shaking	Tank roof damage.	0.25	2.25	0.50	7			1	4
Ground Failure	PGD Failure	1.00				36	0.50	0	5

Notes: (a) Damage Factor is ratio of repair cost of subcomponent / replacement value of component.
(b) Functionality Code: 0 means not functional; 1 means functional.
(c) Logic Codes: For each Logic Code, only one Damage State can occur. For each Logic Code for which demand exceeds capacity, damage state with largest Damage Factor is selected.

Table 5-8. Fragility Curves. Unanchored Redwood Tank. 50,000 to 500,000 Gallons

Wood Tanks – Elevated. A few of these tanks are in use today in major water systems. ATC-13 [ATC] and empirical data suggests that elevated tanks are more vulnerable than tanks at grade. Use Table 5-8 with medians reduced by 25%.

Steel Tanks at Grade – Unanchored. Use Table 5-10 for smaller tanks under 2,000,000 gallons. Use damage algorithm Table 5-13 for larger tanks over 2,000,000 gallons. If the tank is known to have a wooden roof, add damage state Table 5-11(4) for smaller tanks, or Table 5-14(2) for larger tanks with roof damage. If the tank does not have a wooden roof, exclude these damage states.

Steel Tanks at Grade – Anchored. Use Table 5-12 for smaller tanks under 2,000,000 gallons. Use Table 5-14 for larger tanks over 2,000,000 gallons. If the tank is known to have a wooden roof, add damage state Table 5-11(4) for smaller tanks, or T able 5-14(2) for larger tanks with roof damage. If the tank does not have a wooden roof, exclude these damage states.

Hazard	Damage State	Damage Factor	Ground Shaking Median A (g)	Ground Shaking Beta	Ground Shaking Freq. (Hz)	Ground Failure Median PGD (inch)	Ground Failure Beta	Functionality Code	Logic Code
Ground Shaking	Weld failure at base. Loss of contents	0.025	3.00	0.50	8			0	1
Ground Shaking	Pipe damage. Loss of contents	0.007	1.80	0.50	8			0	2
Ground Shaking	Pipe damage. Slight Leakage	0.003	1.00	0.50	8			1	2
Ground Shaking	Elephant foot buckle with loss of contents	1.00	1.00	0.50	8			0	3
Ground Shaking	Elephant foot buckle with no leak	0.50	0.75	0.50	8			1	3
Ground Failure	PGD Failure	1.00				36	0.50	0	4

Notes: (a) Damage Factor is ratio of repair cost of subcomponent / replacement value of component.
(b) Functionality Code: 0 means not functional; 1 means functional.
(c) Logic Codes: For each Logic Code, only one Damage State can occur. For each Logic Code for which demand exceeds capacity, damage state with largest Damage Factor is selected.

Table 5-10. Fragility Curves. Unanchored Steel Tank. 100,000 to 2,000,000 Gallons

Hazard	Damage State	Damage Factor	Ground Shaking Median A (g)	Ground Shaking Beta	Ground Shaking Freq. (Hz)	Ground Failure Median PGD (inch)	Ground Failure Beta	Functionality Code	Logic Code
Ground Shaking	Weld failure at base. Loss of contents	0.025	3.00	0.50	8			0	1
Ground Shaking	Pipe damage/sliding. Loss of contents	0.007	1.80	0.50	8			0	2
Ground Shaking	Pipe damage/uplift. Slight leakage	0.003	1.00	0.50	8			1	2
Ground Shaking	Roof damage / sloshing	0.20	0.50	0.55	0.28			1	3
Ground Shaking	Elephant foot buckle with loss of contents	1.00	1.00	0.50	8			0	4
Ground Shaking	Elephant foot buckle with no leak	0.50	0.75	0.50	8			1	4
Ground Failure	PGD Failure	1.00				36	0.50	0	5

Notes: (a) Damage Factor is ratio of repair cost of subcomponent / replacement value of component.
(b) Functionality Code: 0 means not functional; 1 means functional.
(c) Logic Codes: For each Logic Code, only one Damage State can occur. For each Logic Code for which demand exceeds capacity, damage state with largest Damage Factor is selected.

Table 5-11. Fragility Curves. Unanchored Steel Tank. Wood Roof. 100,000 to 2,000,000 Gallons

Hazard	Damage State	Damage Factor	Ground Shaking Median A (g)	Ground Shaking Beta	Ground Shaking Freq. (Hz)	Ground Failure Median PGD (inch)	Ground Failure Beta	Functionality Code	Logic Code
Ground Shaking	Weld failure at base. Loss of contents	0.03	5.70	0.50	7			0	1
Ground Shaking	Pipe damage/sliding. Loss of contents	0.04	4.00	0.50	7			0	2
Ground Shaking	Pipe damage/uplift. Slight leak.	0.02	3.60	0.50	7			1	2
Ground Shaking	Anchor damage, no leak	0.003	3.10	0.50	7			1	2
Ground Shaking	Elephant foot buckle with loss of contents	1.00	5.55	0.50	7			0	3
Ground Shaking	Elephant foot buckle with no leak	0.50	3.70	0.50	7			1	3
Ground Failure	PGD Failure.	1.00				36	0.50	1	4

Notes: (a) Damage Factor is ratio of repair cost of subcomponent / replacement value of component.
(b) Functionality Code: 0 means not functional; 1 means functional.
(c) Logic Codes: For each Logic Code, only one Damage State can occur. For each Logic Code for which demand exceeds capacity, damage state with largest Damage Factor is selected.

Table 5-12. Fragility Curves. Anchored Steel Tank. 100,000 to 2,000,000 Gallons

Hazard	Damage State	Damage Factor	Ground Shaking Median A (g)	Beta	Freq. (Hz)	Ground Failure Median PGD (inch)	Beta	Functionality Code	Logic Code
Ground Shaking	Weld failure at base. Loss of contents	0.006	2.10	0.50	5			0	1
Ground Shaking	Pipe damage/uplift. Loss of contents	0.002	1.40	0.50	5			0	2
Ground Shaking	Pipe damage/uplift. Slight leak.	0.0006	1.20	0.50	5			1	2
Ground Shaking	Elephant foot buckle with loss of contents	1.00	0.75	0.50	5			0	3
Ground Shaking	Elephant foot buckle without leak.	0.50	0.50	0.50	5			1	3
Ground Shaking	Hoop overstress	0.04	0.95	0.50	5			1	4
Ground Failure	PGD Failure.	1.00				36	0.50	0	5

Notes: (a) Damage Factor is ratio of repair cost of subcomponent / replacement value of component.
(b) Functionality Code: 0 means not functional; 1 means functional.
(c) Logic Codes: For each Logic Code, only one Damage State can occur. For each Logic Code for which demand exceeds capacity, damage state with largest Damage Factor is selected.

Table 5-13. Fragility Curves. Unanchored Steel Tank. > 2,000,000 Gallons

Hazard	Damage State	Damage Factor	Ground Shaking Median A (g)	Beta	Freq. (Hz)	Ground Failure Median PGD (inch)	Beta	Functionality Code	Logic Code
Ground Shaking	Pipe damage / uplift. Loss of contents	0.007	3.20	0.50	5			0	1
Ground Shaking	Roof Damage.	0.08	0.20	0.50	0.13			1	2
Ground Shaking	Hoop overstress	0.11	4.10	0.50	5			1	3
Ground Shaking	Anchor damage, weld failure at base. Loss of contents	0.002	3.60	0.50	5			0	4
Ground Failure	PGD Failure.	1.00				36	0.50	0	5

Notes: (a) Damage Factor is ratio of repair cost of subcomponent / replacement value of component.
(b) Functionality Code: 0 means not functional; 1 means functional.
(c) Logic Codes: For each Logic Code, only one Damage State can occur. For each Logic Code for which demand exceeds capacity, damage state with largest Damage Factor is selected.

Table 5-14. Fragility Curves. Anchored Steel Tank. Wood Roof. >2,000,000 Gallons

Steel Tanks – Elevated. ATC-13 and limited empirical data suggest that elevated tanks are more vulnerable than tanks at grade. The typical failure mode is collapse. Insufficient empirical data exists to construct an empirically based damage algorithm. The following is assumed:

- The tanks are always designed for wind load, which can be approximated at about equivalent to a PGA of 0.03g. In Zone 3/4, the tanks have been originally designed elastically for a PGA of 0.15g. There should be essentially no failures at this level of shaking.
- The median collapse fundamental mode for Spectral Acceleration shown in Table 5-16 is 0.7g for tanks not designed for earthquake loading. The median collapse fundamental mode Spectral Acceleration shown in Table 5-17 is 1.0g for tanks designed for nominal, not site-specific, earthquake loads in Zone 3/4.

Hazard	Damage State	Damage Factor	Ground Shaking Median A (g)	Ground Shaking Beta	Ground Shaking Freq. (Hz)	Ground Failure Median PGD (inch)	Ground Failure Beta	Functionality Code	Logic Code
Ground Shaking	Collapse	1.00	0.70	0.55	1.5			0	1
Ground Failure	PGD Failure.	1.00				24	0.50	0	2

Notes: (a) Damage Factor is ratio of repair cost of subcomponent / replacement value of component.
(b) Functionality Code: 0 means not functional; 1 means functional.
(c) Logic Codes: For each Logic Code, only one Damage State can occur. For each Logic Code for which demand exceeds capacity, damage state with largest Damage Factor is selected.

Table 5-16. Fragility Curves. Elevated Steel Tank. Non Seismic Design

Hazard	Damage State	Damage Factor	Ground Shaking Median A (g)	Ground Shaking Beta	Ground Shaking Freq. (Hz)	Ground Failure Median PGD (inch)	Ground Failure Beta	Functionality Code	Logic Code
Ground Shaking	Collapse	1.00	1.00	0.55	1.5			0	1
Ground Failure	PGD Failure.	1.00				24	0.50	0	2

Notes: (a) Damage Factor is ratio of repair cost of subcomponent / replacement value of component.
(b) Functionality Code: 0 means not functional; 1 means functional.
(c) Logic Codes: For each Logic Code, only one Damage State can occur. For each Logic Code for which demand exceeds capacity, damage state with largest Damage Factor is selected.

Table 5-17. Fragility Curves. Elevated Steel Tank. Nominal Seismic Design

- The typical fundamental frequency of these tanks is 1 to 2 Hz or about 1.5 Hz. On rock sites, spectral acceleration at 1.5 Hz is about the same as the PGA. On soil sites, spectral acceleration at 1.5 Hz is about 2 times that of the PGA.
- Assume $\beta = 0.3$. This reflects uncertainty in the tank capacity.
- For elevated tanks with no seismic design, assume:
  - Median Acceleration to failure = 0.70g
  - Fundamental frequency = 1.5 Hz
  - Damage Factor = 100%
  - Functionality Factor = 0 (not functional)

This damage algorithm translates to the following failure rates for elevated steel tanks on rock sites:

- 50% of elevated tanks fail at PGA = 0.70g
- 16% of elevated tanks fail at PGA = 0.38g
- 2.3% of elevated tanks fail at PGA = 0.21g
- 0.13% of elevated tanks fail at PGA = 0.12g

These failure rates would occur at half the PGAs for elevated tanks on soil sites. These damage algorithms appear reasonable, given the limited empirical evidence available.

For elevated tanks with nominal seismic design, assume:

- Median Acceleration to failure = 1.0g
- Fundamental frequency = 1.5 Hz

- Damage Factor = 100%
- Functionality Factor = 0 (not functional)

This damage algorithm translates into the following failure rates for elevated steel tanks on rock sites:

- 50% of elevated tanks fail at PGA = 1.0g
- 16% of elevated tanks fail at PGA = 0.55
- 2.3% of elevated tanks fail at PGA = 0.30
- 0.13% of elevated tanks fail at PGA = 0.17

These failure rates would occur at half the PGAs for elevated tanks on soil sites. These damage algorithms appear reasonable, given the limited empirical evidence available.

Elevated tanks with site-specific seismic design should have less than a 2% chance of failure for the design basis event. No damage algorithm is provided for these types of tanks.

To summarize:

- Use Table 5-16 for elevated steel tanks with no seismic design.
- Use Table 5-17 for elevated steel tanks with nominal seismic design.

Concrete Tanks At Grade - Unanchored. Use Table 5-9 for unanchored prestressed concrete tanks at grade. For larger volume concrete tanks, use Table 5-9 without modification.

Hazard	Damage State	Damage Factor	Ground Shaking Median A (g)	Ground Shaking Beta	Ground Shaking Freq. (Hz)	Ground Failure Median PGD (inch)	Ground Failure Beta	Functionality Code	Logic Code
Ground Shaking	Uplift of Wall. Slight Leakage	0.015	2.00	0.45	7			1	1
Ground Shaking	Cracking of Tank Wall. Loss of Contents	0.100	1.05	0.45	7			0	2
Ground Shaking	Sliding of tank wall. Slight Leakage	0.015	0.25	0.45	7			1	3
Ground Shaking	Major hoop over-stress. Loss of Contents	0.030	0.75	0.45	7			0	4
Ground Shaking	Slight hoop over-stress. Slight Leakage	0.015	0.45	0.45	7			1	4
Ground Shaking	Roof Failure	0.040	2.60	0.45	7			1	5
Ground Failure	PGD failure	1.000				24	0.50	0	6

Notes: (a) Damage Factor is ratio of repair cost of subcomponent / replacement value of component.
(b) Functionality Code: 0 means not functional; 1 means functional.
(c) Logic Codes: For each Logic Code, only one Damage State can occur. For each Logic Code for which demand exceeds capacity, damage state with largest Damage Factor is selected.

Table 5-9. Fragility Curves. Unanchored Concrete Tank. >1,000,000 Gallons

Concrete Tanks At Grade - Anchored. Use Table 5-15 for anchored reinforced concrete tanks at grade. Insufficient empirical evidence exists on how modern, seismically designed, prestressed concrete tanks have performed in earthquakes. Assume they will perform as well as anchored reinforced concrete tanks using Table 5-15. For larger volume concrete tanks, use Table 5-15 without modification.

Hazard	Damage State	Damage Factor	Ground Shaking Median A (g)	Beta	Freq. (Hz)	Ground Failure Median PGD (inch)	Beta	Functionality Code	Logic Code
Ground Shaking	Uplift - Crush Concrete	0.10	1.30	0.50	9			0	1
Ground Shaking	Sliding	0.03	1.10	0.50	9			0	2
Ground Shaking	Shearing of Tank Wall	0.03	1.60	0.50	9			0	3
Ground Shaking	Hoop Overstress	0.03	4.10	0.50	9			1	4
Ground Failure	PGD Failure.	0.75				24	0.50	0	5

Notes: (a) Damage Factor is ratio of repair cost of subcomponent / replacement value of component.
(b) Functionality Code: 0 means not functional; 1 means functional.
(c) Logic Codes: For each Logic Code, only one Damage State can occur. For each Logic Code for which demand exceeds capacity, damage state with largest Damage Factor is selected.

Table 5-15. Fragility Curves. Anchored Concrete Tank. 50,000 to 1,000,000 Gallons

Concrete Tanks – Elevated. Although not common in the US, elevated reinforced concrete tanks are used in other countries. More than 100 of such tanks have been exposed to moderate to strong ground shaking in recent earthquakes including those in Kocaeli, Turkey in 1999 and in Gujarat, India in 2001. While complete design details for these tanks are not available, it is believed they were designed to seismic forces about equivalent to those specified in UBC (1994 version) for seismic zone 3 to 4. Observed performance of these tanks suggests that they undergo moderate damage such as spalling of concrete columns at joints, at PGA levels of about 0.2 to 0.3g, and have a less than 5% chance of collapse at PGA levels of about 0.4 to 0.5g.

Open Cut Reservoirs. Damage to open cut reservoirs without roofs is generally limited to failure of embankment dams. Fragility curves for dams are not covered in this report.

Damage to open cut reservoirs with roofs depends on the type of roof. Most open cut reservoir roofs were installed in the 1960s or earlier, and many of these are considered highly vulnerable to strong seismic forces.

Although damage to these roofs should not impair reservoir performance—it will still hold water and it is assumed that falling debris will not clog the inlet/outlet pipes—damage will affect water quality (e.g., debris in the water) and will cause large financial losses, as repairing the roofs can be very expensive.

Use Table 5-18 for open cut reservoir roofs with little or no seismic design (e.g., under 0.10g equivalent static force). Note that functional failure of the reservoir depends on failure of the embankment dams, which is not covered by Table 5-18.

Hazard	Damage State	Damage Factor	Ground Shaking Median A (g)	Beta	Freq. (Hz)	Ground Failure Median PGD (inch)	Beta	Functionality Code	Logic Code
Ground Shaking	Major Damage to Roof	0.15	1.00	0.55	4			1	1
Ground Shaking	Minor Damage to Roof	0.05	0.60	0.55	4			1	2
Ground Failure	Localized Damage to Roof	0.05				8	0.50	1	3

Notes: (a) Damage Factor is ratio of repair cost of subcomponent / replacement value of component.
(b) Functionality Code: 0 means not functional; 1 means functional.
(c) Logic Codes: For each Logic Code, only one Damage State can occur. For each Logic Code for which demand exceeds capacity, damage state with largest Damage Factor is selected.

Table 5-18. Fragility Curves. Open Cut Reservoir Roof

Fiberglass Tanks. Water utilities commonly use fiberglass tanks to store caustic materials at water treatment plants. Significant seismic weaknesses may be present at locations where the tanks are attached or anchored to foundations. This report provides no fragility data for such tanks. Seismic anchor systems for these tanks are warranted in many situations. New ones should be designed and existing ones should be verified by a licensed structural engineer.

5.4.1 Use of Fault Trees for Overall Tank Evaluation

One method to determine whether a tank is in a particular damage state is the use of a fault tree. The calculation procedure to determine the functional status of a tank for a scenario earthquake is as follows:

- Determine the PGA, Response Spectra and PGD at the tank site.
- Determine the functional status of each of the lowest level component items at the tank. These appear at the lowest level in the fault tree. Figure 5-1 gives an example for a steel tank with eight possible failure modes that can be combined into one of three possible overall tank damage states.
- The probability of failure of each lowest level component should be determined. For example, assume that a component has a single damage state, namely: "Roof damaged, median A = 0.50g, beta = 0.20. Also assume that a Spectral Acceleration at this site is at the fundamental frequency of this component of 0.41g. Then, by assuming a lognormal distribution for the fragility curve, the probability of failure of this component is:
  - $P_f\,(SA = 0.41) = A\ e^{-x\,\beta}$,
  - $0.41 = 0.50\ \ e^{\ x\,(0.20)}$
    - $x = \ln(\frac{0.41}{0.50})\ /\ (0.20) = -1.00$
  - x = -1.00, or one standard deviation below the median. Using standard normal tables, we find that 1.00 standard deviation below the median means that there is a 16% chance that the actual components capacity is less than 0.41g.
- Once the component-level failure probabilities are determined for each of the eight failure modes, the logic of the fault trees (three are shown in Figure 5-1) is calculated to determine the probability of failure of the three highest level events (or damage states 2, 3 and 4 in Figure 5-1). The method to handle fault tree logic is as follows:
  - The event above an And Gate is computed as follows when there are n components below the And Gate:

$$P_{f\ \text{Event above And gate}} = \prod_{i=1}^{n} [P_f]_{\text{Components below And gate}}$$

- The event above an Or Gate is computed as follows when there are n components below the Or Gate:

$$P_{f\ \text{Event above Or gate}} = 1 - \prod_{i=1}^{n} [1-P_f]_{\text{Components below Or gate}}$$

- Following this procedure, the probability of each of the top events occurring can be obtained; namely, the probability that the tank is in each of the three damage states.

For the example in Figure 5-1, the results are as follows: Probability of being in Damage State 2 = 50%, Damage State 3 = 10%, Damage State 4 = 30%. Note that the lowest level events may not be mutually exclusive. For example, “Wall Uplift with Leak” implies that “Anchor Bolts Damaged” also occurs. The manner in which the fault trees are constructed should reflect the ways in which the results will be used. A different fault tree might be used for estimating the total repair cost for the tank, as compared as to evaluating whether or not the tank remains functional immediately after the earthquake.

5.5 References

API, American Petroleum Institute, *Welded Steel Tanks for Oil Storage, API Standard 650*, Washington, DC.

AWWA, American Water Works Association, *Welded Steel Tanks for Water Storage, AWWA Standard D100*, Denver, Colorado.

ATC, *Earthquake Damage Evaluation Data for California*, ATC 13, Applied Technology Council, 1985.

Ballantyne, D., Crouse, C., *Reliability and Restoration of Water Supply Systems for Fire Suppression and Drinking Following Earthquakes*, prepared for National Institute of Standards and Technology, NIST GCR 97-730, 1997.

Bandyopadhyay, K., Cornell, A., Costantino, C., Kennedy, R.P., Miller, C. and Veletsos, A., *Seismic Design and Evaluation Guidelines for The Department of Energy high-Level Waste Storage Tanks and Appurtenances*, Brookhaven National Laboratory, BNL 52361, Jan. 1993.

Brown, K., Rugar, P., Davis, C., Rulla, T., "Seismic Performance of Los Angeles Water Tanks, Lifeline Earthquake Engineering," in *Proceedings, 4th US Conference*, ASCE, TCLEE Monograph No. 6, San Francisco, California, 1995.

Cooper, T. W., *A Study of the Performance of Petroleum Storage Tanks During Earthquakes, 1933-1995*, prepared for the National Institute of Standards and Technology, Gaithersburg, MD, June 1997.

EERI, *Imperial County, California Earthquake Reconnaissance Report*, Earthquake Engineering Research Institute, G, Brandow, Coordinator, D. Leeds, Editor, Feb. 1980.

EERI, *Coalinga, California Earthquake of May 2, 1983 Reconnaissance Report*, Earthquake Engineering Research Institute, R. Scholl, J. Stratta, Editors, Jan. 1984.

EERI, *Morgan Hill*, *California Earthquake of April 24, 1984*, Earthquake Engineering Research Institute, Vol. 1, No. 3, May, 1985.

EERI, "*Loma Prieta Reconnaissance Report*," *Earthquake Spectra*, Earthquake Engineering Research Institute, Supplement to Vol. 6, May 1990.

EERI, "Costa Rica Earthquake Reconnaissance Report," *Earthquake Spectra*, Earthquake Engineering Research Institute, Supplement B to Volume 7, Oct. 1991.

EERI, "Northridge Earthquake Reconnaissance Report," *Earthquake Spectra*, Earthquake Engineering Research Institute, Supplement C to Volume 11, Vol. 1. April 1995.

Eidinger, J. and Avila, E. eds., *Seismic Design and Retrofit of Water Transmission Facilities,* TCLEE Monograph No. 15, ASCE, Jan. 1999.

Hanson, R., *The Great Alaska Earthquake of 1964*, National Academy of Sciences, 1973.

Haroun, M., "Behavior of Unanchored Oil Storage Tanks: Imperial Valley Earthquake," *Journal of Technical Topics in Civil Engineering*, ASCE, Vol. 109, No. 1, April 1983.

Haroun, M. A., "Implications of Recent Nonlinear Analyses on Seismic Standards of Liquid Storage Tanks," in *Proceedings, 5th US Conference on Lifeline Earthquake Engineering*, TCLEE Monograph No. 16, ASCE, Seattle, Aug. 1999.

Hashimoto, P.S., and Tiong, L.W., *Earthquake Experience Data on Anchored Ground Mounted Vertical Storage Tanks*, EPRI Report NP-6276, March 1989.

HAZUS, *Earthquake Loss Estimation Methodology*, National Institute of Building Sciences, prepared by Risk Management Solutions, Menlo Park, CA, 1997.

Kennedy, R. P. and Kassawara, R. P., "Seismic Evaluation of Large Flat-Bottomed Tanks," *Second Symposium on Current Issues Related to Nuclear Power Plant Structures, Equipment, and Piping with Emphasis on Resolution of Seismic Issues in Low-Seismicity Regions*, EPRI NP-6437-D, May 1989.

NZNSEE, *Seismic Design of Storage Tanks, Recommendations of a Study Group of the New Zealand National Society for Earthquake Engineering*, Dec. 1986.

O'Rourke, M. and So, P., "Seismic Behavior of On-Grade Steel Tanks; Fragility Curves," in *Optimizing Post-Earthquake Lifeline System Reliability*, ed. W. Elliott and P. McDonough, Technical Council on Lifeline Earthquake Engineering, Monograph No. 16, ASCE, Aug. 1999.

Wald, J., Quitoriano, V., Heaton, T., Kanamori, H., and Scrivner, C., TRINET Shakemaps: Rapid Generation of Peak Ground Motion and Intensity Maps for Earthquakes in Southern California, Earthquake Spectra, Oct. 1998.

5.6 Figures

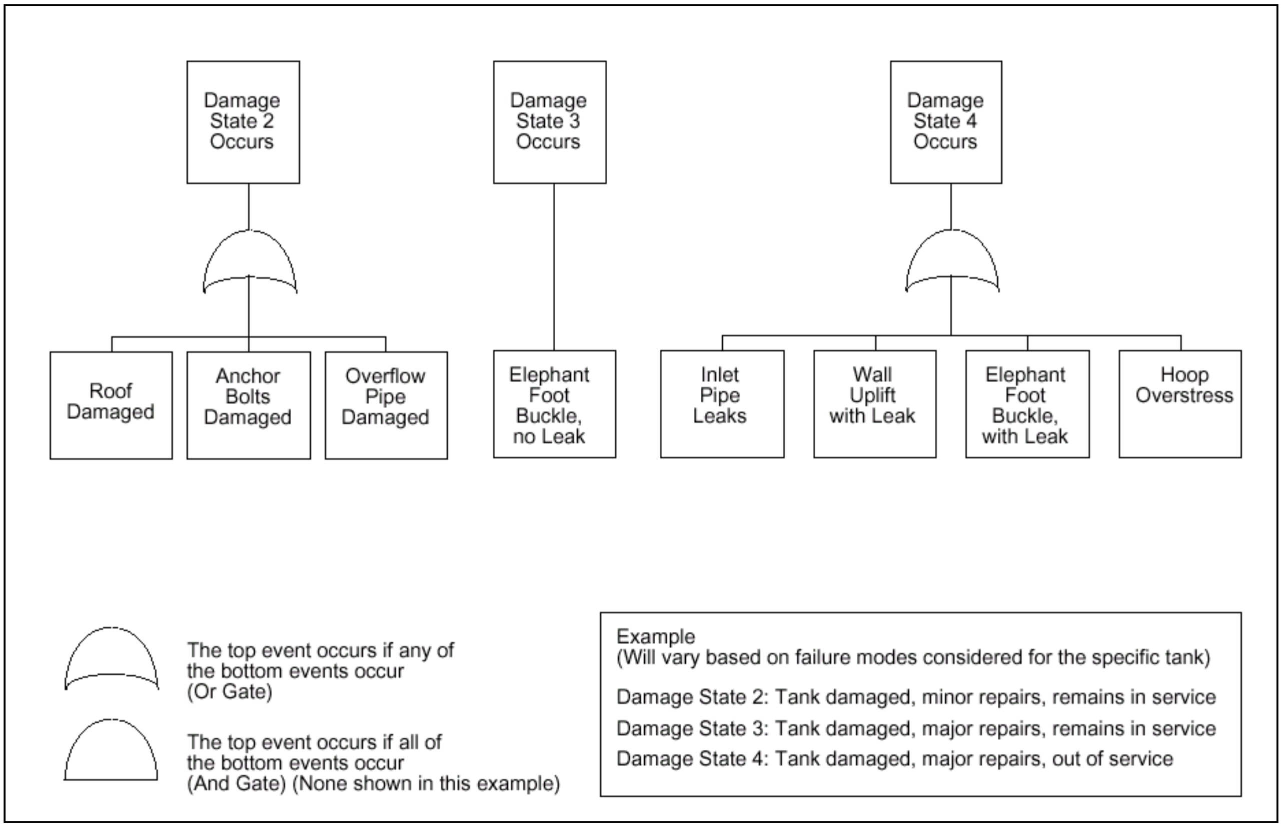


Figure 5-1. Example Fault Trees for Evaluation of an Anchored Steel Tank

6.0 Water Tunnel Fragility Formulations

Section 6 of the report provides fragility curves for tunnels in response to strong ground shaking. In some instances, damage to tunnels due to landslides and surface faulting is discussed, but the development of fragility curves for these damage modes is best done using tunnel-specific calculations, which are beyond the scope of this report.

6.1 Factors That Cause Damage to Tunnels

Water tunnels may be damaged in earthquakes due to ground shaking, landslides or fault offset. This report assumes that water tunnels are transporting water at near-atmospheric pressures; i.e., tunnels not designed to retain high internal pressures.

Ground shaking will induce stresses in the liner system of tunnels. If the level of shaking is sufficiently high and, depending on the type and quality of the liner system, the liner can become cracked. With sufficient cracking, some parts of the liner can collapse into the tunnel. For unlined tunnels, ground shaking can cause similar failure of the native materials.

For water tunnels, the impact of liner failure may or may not be immediate. Small cracks in liners will not generally directly impact the flow of water through the tunnel, although there may be some minor increases in head loss. Over time, small cracks will allow water from the tunnel to enter the native materials behind the liner, which could cause erosion of the materials and ultimately could lead to more damage to the liner. For this reason, even with minor damage, water utilities will often take the tunnel out of service and repair the liner.

Large cracks in liners, considered to be moderate damage, could lead to immediate impacts to the tunnel. Large dropouts of the liner into the tunnel could lead to a partial blockage of water flow, or carry liner debris or native material debris in the water, which could impact downstream water quality or damage in-line equipment like pumps. A tunnel with moderate damage might be operable for days or even months following an earthquake; not repairing moderate damage could lead to a failure of the tunnel over time.

Major damage to liner systems could lead to an immediate stop of all or almost all flow of water through the tunnel.

For the most part, the factors that lead to the major damage state are fault offset through the tunnel itself or landslide at the tunnel portals. This report does not provide for fragility of tunnels due to landslide or fault offset, but Appendix C gives some data on these failure modes.

6.2 Empirical Tunnel Dataset

Table C-2 presents a database of 217 bored tunnels that have experienced strong ground motions in prior earthquakes. Table C-2 is composed of 204 entries based on work by Power et al [1998] and supplemented by case history data based on Asakura and Sato [1998]. The database is described in detail in Appendix C.

Table 6-1 summarizes the performance of the first 204 entries from Table C-2. This includes a total of 204 observations from moderate-to-large magnitude earthquakes or those with a magnitude range M_w 6.6 to 8.4. Of these 204 cases, 97 are from the 1995 Kobe, Japan earthquake, for which a detailed compilation of tunnel performance data was made by the Japanese Geotechnical Engineering Association [1996]. The next largest contributors to the database are the 1994 Northridge and 1989 Loma Prieta earthquakes, with 31 and 22 cases, respectively. The database includes tunnels built for various functions (i.e., highway, transit, railroad, water supply and communications). Most of the observations are for railroad and water supply tunnels and most data for highway tunnels is from the 1995 Kobe earthquake.

Earthquake	M_w	Unknown Liner	Unlined	Timber or Masonry Liner	Concrete Liner	Reinforced Concrete or Steel Pipe Liner	Total
1906 San Francisco, CA	7.8	–	1	7	–	–	8
1923 Kanto, Japan	7.9	–	7	4	2	–	13
1952 Kern County, CA	7.4	–	4	–	–	–	4
1964 Alaska	8.4	–	8	–	–	–	8
1971 San Fernando, CA	6.6	–	8	–	–	1	9
1989 Loma Prieta, CA	7.1	3	–	2	11	6	22
1992 Petrolia, CA	6.9	–	–	–	11	–	11
1993 Hokkaido, Japan	7.8	–	–	–	–	1	1
1994 Northridge, CA	6.7	6	–	–	5	20	31
1995 Kobe, Japan	6.9	3	–	1	87	6	97
TOTAL							**204**

Table 6-1. Summary of Earthquakes and Lining/Support Systems of the Bored Tunnels in the Database in Table C-2 [after Power et al, 1998]

6.3 Tunnel Fragility Curves

Table C-2 can be used to determine the percentage of tunnels of a given class of construction experiencing defined damage states during different levels of shaking. Table 6-2 provides a breakdown of the tunnels in the database. For tunnels with multiple types of liners, the tunnel was classified according to the “best” type of liner system anywhere along the length of the tunnel. The four damage states are: DS=1 none; DS=2 slight; DS=3 moderate; and DS=4 heavy.

PGA (g)	All Tunnels	DS = 1	DS = 2	DS = 3	DS = 4
0.07	30	30	0	0	0
0.14	19	18	1	0	0
0.25	22	19	2	0	1
0.37	15	14	0	0	1
0.45	44	36	6	2	0
0.57	66	44	12	9	1
0.67	19	3	7	8	1
0.73	2	0	0	2	0
Total	**217**	**164**	**28**	**21**	**4**

Table 6-2. Statistics for All Bored Tunnels in Table C-2

Table 6-3 presents the computed fragilities for bored tunnels based on the data in Table 6-2. See Appendix C for further breakdown of the data.

DS	A, g	Beta	A, g	Beta	A, g	Beta	A, g	Beta	A, g	Beta
DS≥2	0.60	0.11	0.33	0.21	0.43	0.03	0.61	0.10	0.61	0.27
DS≥3	0.65	0.12	0.55	0.39	0.57	0.01	0.67	0.11	0.82	0.34
DS=4										
	All N=217		Unlined N=28		Timber, Masonry, Brick N=14		Unreinforced Concrete N=125		Reinforced Concrete, Steel N=38	

Table 6-3. Fragility Curves, Tunnels, As a Function of Liner System

The fragility curves developed by regression analysis are considered to be a better way of describing the entire data set for use in programs like HAZUS, in that the fragility represents reaching *or exceeding* a particular damage state.

The fragilities in Table 6-3 are described in terms of the median PGA to reach or exceed a particular damage state, and the lognormal standard deviation of the fragility or beta. Since essentially all PGA values in the statistics have been back-calculated at the tunnel location using attenuation models, the beta value represents uncertainty in the ground motion and in the tunnel performance.

Table 6-4 compares the statistics for the complete 217 bored tunnel database (Table 6-1) with the statistics from prior studies [HAZUS 1997] for comparable tunnels.

Good Quality (Reinforced Concrete/Steel) Tunnel	**217 Tunnels Median - PGA**	**HAZUS Median - PGA**
Moderate Damage	0.82 g	0.8 g
Minor Damage	0.61 g	0.6 g

Poor to Average Quality (Unreinforced Concrete, Timber, Masonry, Unlined) Tunnel	**217 Tunnels Median - PGA**	**HAZUS Median - PGA**
Moderate Damage	0.55 to 0.67 g	0.7 g
Minor Damage	0.33 to 0.61 g	0.5 g

Table 6-4. Comparison of Bored Tunnel Fragility Curves

The comparisons in Table 6-4 suggest the following:

- For bored tunnels with reinforced concrete or steel liners, the database shows a median of 0.61g for the minor damage state. The corresponding HAZUS value is 0.6g, which was based on engineering judgment.
- For bored tunnels with unreinforced concrete, timber or masonry liners, or for unlined tunnels, the database shows a median of between 0.33 and 0.61g for the minor damage state. The corresponding HAZUS value is 0.5g, which was based on engineering judgment. The database shows a median of 0.55 to 0.67 for the moderate damage state and the corresponding HAZUS value is 0.7g, which was based on engineering judgment. This suggests it is appropriate to slightly modify and lower the HAZUS median PGA value for the minor and moderate damage states.

The tunnel damage data in Tables C-3 and C-5 could not be directly included in the complete database (Table C-2) because of many missing attributes. The information in Table C-3 could be refined in future studies into a format more compatible with Table C-2 to allow statistical analysis. The data in Table C-5 is focused only on the moderate to heavy damage states, and the following observations are made:

- For the Japanese earthquake data tabulated in Table C-5, 16 of the case histories are from the 1923 Kanto earthquake. Data was compared in Table C-2 with the 13 case histories summarized by Power et al. [1998] for the 1923 Kanto earthquake. The compilation of tunnel damage reported in Table C-5 is similar, but not the same as that in Table C-2; however, the net effect would not significantly change the number of tunnels experiencing moderate to heavy damage during this earthquake.
- For the other 18 case histories of seismic tunnel performance in Japan tabulated in Table C-5, most cases of moderate to heavy damage may be associated with landsliding, faulting, other forms of ground failure, or with tunnels under construction at the time of the earthquake. This is noted in Table C-5. For most other tunnels in these Japanese earthquakes, damage was apparently slight or none.

The tunnels in Turkey that collapsed from the Duzce 1999 earthquake were under construction and thus should not be included in fragility assessments for completed tunnels.

Given the analysis of the all the information available about tunnels that have suffered the complete damage state, the following observations are made:

- Four such tunnels out of 217 in Table C-2 are denoted with Damage Mode = 4. Of these four, three reached the “heavy” damage state because of landslide or surface faulting, coupled with poor quality construction and poor geologic conditions.
- One highway tunnel, shown as entry 33 in Table C3-5, reached the heavy damage state (excluding the 1923 Kanto earthquake), at a location in the main liner section sufficiently far from the portal. This tunnel was located 26 km from the epicenter of the magnitude 6.8 Noto offshore earthquake. It was a 76-meter long and 6-meter wide road tunnel. About 16 meters of the liner collapsed in the center of the tunnel, forcing the tunnel out of service. Kunita et al report the following reasons for the liner collapse:
    - The ground consisted of alternating layers of soft turf and mudstone and was subject to loosening.
    - The loosened areas around the tunnels had expanded as ground deterioration progressed over a long period of time under the influence of weathering and ground water, some 31 years between construction and the earthquake. Voids already existed behind the concrete lining and in the surrounding ground.
    - Loosened areas around new openings created by falling and around soft areas had expanded under the influence of the earthquake.
    - The earthquake-induced impulsive earth pressures and asymmetrical pressures on the concrete lining caused the collapse of the arch of the concrete lining and of the ground directly above the arch.

Given the available information, it would appear reasonable to make the following statements about the potential for tunnel collapse due to ground shaking:

- No such failures have occurred to well-constructed tunnels in good ground conditions.
- Perhaps as few as four such failures have occurred in tunnels with either unreinforced concrete, timber or masonry liners in poor ground conditions. High levels of PGA have not been attributed to such failures. For purposes of establishing a fragility level for this damage state, it is assumed that 1 in 100 tunnels with these attributes will experience such failure at PGA levels of about 0.35g. Allowing a beta of 0.5, then the back-calculated median PGA is 1.12g. Using this description of the fragility, the chance of a tunnel collapse is 17% at a PGA = 0.7g for similar conditions. Using this fragility curve to predict the heavy damage state would be useful for preliminary loss estimation purposes only.

Using the above findings as a guide, judgments were made regarding median values of PGA at ground surface at outcropping rock for the damage categories of slight, moderate and heavy. Slight damage includes minor cracking and spalling and other minor distress to tunnel liners. Moderate damage ranges from major cracking and spalling to rock falls. Heavy damage includes collapse of the liner or surrounding soils to the extent that the tunnel is blocked either immediately or within a few days after the main shock. These assessments are made for tunnels in rock and tunnels in soil, in both poor-to-average construction and conditions and in good construction and conditions.

Rock Tunnels with poor-to-average construction and conditions. Tunnels in average or poor rock, either unsupported masonry or timber liners, or unreinforced concrete with frequent voids behind lining and/or weak concrete.

Rock Tunnels with good construction and conditions. Tunnels in very sound rock and designed for geologic conditions (e.g., special support such as rock bolts or stronger liners in weak zones); unreinforced, strong concrete liners with contact grouting to assure continuous contact with rock; average rock; or tunnels with reinforced concrete or steel liners with contact grouting.

Alluvial (Soil) and Cut and Cover Tunnels with poor to average construction. Tunnels that are bored or cut and cover box-type tunnels and include tunnels with masonry, timber or unreinforced concrete liners, or any liner in poor contact with the soil. These also include cut and cover box tunnels not designed for racking mode of deformation.

Alluvial (Soil) and Cut and Cover Tunnels with good construction. Tunnels designed for seismic loading, including racking mode of deformation for cut and cover box tunnels. These also include tunnels with reinforced strong concrete or steel liners in bored tunnels in good contact with soil.

The assessed values of PGA for these damage states and tunnel categories are summarized in Table 6-5. Tables 6-6 and 6-7 compare the data in Table 6-5 with the data in Tables C-9 to C-12. The magnitude of the median fragilities are about the same for tunnels of good quality construction and somewhat lower for tunnels of lower quality construction. The estimated dispersion parameter beta is 0.4 for the slight and moderate damage states and 0.5 for the heavy damage state. Beta includes randomness in tunnel performance and uncertainty in ground motion. The heavy damage state is provided only for tunnels with poor-to-average conditions, and with the limitations noted in the text above.

Type of Tunnel (see text for detailed description)	**Slight Damage State Median PGA (g)**	**Moderate Damage State Median PGA (g)**	**Heavy Damage State Median PGA (g)**
Rock Tunnel: poor-to-average construction and conditions	0.35	0.55	1.10
Rock Tunnel: good construction and conditions	0.61	0.82	–
Soil Tunnel: poor-to-average construction	0.30	0.45	0.95
Soil Tunnel : good construction	0.50	0.70	–

Table 6-5. Tunnel Fragility – Median PGAs – Ground Shaking Hazard Only

Tunnel Type / Damage State	**HAZUS (PGA)**	**ALA – Current (PGA)**
ROCK		
Heavy Damage		NA
Moderate Damage	0.80 g	0.82 g
Minor or Slight Damage	0.60 g	0.61 g
CUT & COVER OR ALLUVIAL		
Heavy Damage		NA
Moderate Damage	0.70 g	0.70 g
Minor Damage	0.50 g	0.50 g

Table 6-6. Comparison of Tunnel Fragility Curves (Good Quality Construction)

Tunnel Type / Damage State	**HAZUS (PGA)**	**ALA – Current (PGA)**
ROCK		
Heavy Damage		1.10 g
Moderate Damage	0.70 g	0.55 g
Minor or Slight Damage	0.50 g	0.35 g
CUT & COVER OR ALLUVIAL		
Heavy Damage		0.95 g
Moderate Damage	0.55 g	0.45 g
Minor Damage	0.35 g	0.30 g

Table 6-7. Comparison of Tunnel Fragility Curves
(Poor to Average Quality Construction-Conditions)

6.4 References

Asakura, T., and Sato, Y., "Mountain Tunnels Damage in the 1995 Hyogoken-Naubu Earthquake," QR of RTRI, Vol. 39, No. 1, 1998.

Architectural Institute of Japan, *Disaster Investigation Reports on the 1974 Izu-hanto-oki Earthquake and the 1978 Izu-Oshima-kinkai Earthquake*, pp.183-255, 1980 (in Japanese).

Geotechnical Engineering Association, Committee of Hanshin Disaster Investigation, *Hanshin-Awaji Disaster Investigation Report: Commentary Volume, Section 9, Underground Space*, p. 405-440 March, 1996.

HAZUS, *Earthquake Loss Estimation Methodology, Technical Manual, Volume II*, prepared by the National Institute of Building Sciences for Federal Emergency Management Agency, 1997.

Kunita, M., Takemata, R., and Lai, Y., "Restoration of a Tunnel Damaged by Earthquake," in *Tunneling and Underground Space Technology,* Vol. 9, No. 4, pp. 439-448, 1994.

Kunita, M., et al., "Rehabilitation of Tunnel Damage caused by Noto Peninsula Offshore Earthquake," *Tunnels and Underground*, Vol.24, No.11, pp. 7-13, 1993 (in Japanese).

JSCE, "Investigation Report on The Damages by Taisho 12 (1923) Great Kanto Earthquake (Reprint version)," Vol.2: Railroad, Vol.3: Road, 1984 (in Japanese).

JSCE, "Investigation Report on The Damages by Showa 39 (1964) Niigata Earthquake," pp. 100-103, pp. 128-131, p. 147, pp. 150-151, pp. 175-178, pp. 210-220, 1966 (in Japanese).

JSCE, Investigation Committee of the 1978 Miyagiken-oki Earthquake at Tohoku Branch of the Japan Society of Civil Engineer, "General Report on the 1978 Miyagiken-oki Earthquake," pp. 165-168, 1980 (in Japanese).

JSCE, "Investigation Report on the Damages by 1983 Nihonkai-chubu Earthquake," pp. 422-423, 1986 (in Japanese).

Japan Society for Earthquake Engineering Promotion, "Investigation Report on the 1993 Hokkaido-Nansei-Oki Earthquake," *JSEEP NEWS*, No.133, 1993.

Power, M.S., Rosidi, D., and Kaneshiro, J., Gilstrap, S.D., and Chiou, S.J., *Summary and Evaluation of Procedures for the Seismic Design of Tunnels: Multidisciplinary Center for Earthquake Engineering Research*, *Draft Technical Report*, FHWA Contract No. DTFH61-92-C-0012 Task 112-D-5.3c, Sept. 1998.

7.0 Water Canal Fragility Formulations

A canal will be exposed to the same four types of hazards as other water system components: ground shaking, liquefaction, landslides and fault offset. The liquefaction and landslide hazards must be considered both in terms of external factors affecting the canal, as well as liquefaction or landslide within the canal embankments themselves.

The following inventory information about a canal will usually be required in order to assess the seismic performance of a canal.

- Geographic alignment of the canal. The various lengths of the canal will usually be named "reaches," or sometimes marked by "mileposts." Reaches are usually associated with specific in-line hydraulic function of the canal (such as Reach 1, from pump station 1 to turnout 3, etc.). Since earthquake hazards are not usually confined to the boundaries for each reach, the canal will usually need to be discretized in shorter intervals to reflect the varying earthquake hazards.
- Cross sectional shape of the canal, as it varies along the length. Mark each change in station where the design of the canal changes from lined to unlined, in cross-sectional shape, in materials used for embankments, etc.
- Location and type of in-line components, such as intake structures, pump stations and control gates. Seismic evaluation of these components using fragility formulations is beyond the scope of this report.
- Location and type of siphons. Analysis of pipeline and tunnel siphons is covered in Sections 4 and 6 of this report. Short lengths of pipeline siphons, often with special boundary conditions, might best be analyzed using strength-of-material formulations rather than strictly relying on the empirical fragility formulations in Sections 4 and 6. This is because the pipeline empirical fragility formulations of Section 4 are best suited to tens to hundreds of miles of pipeline.
- Location and type of flumes, if any. The seismic evaluation of flumes is not covered in this report. See Knarr for examples for examples of the seismic analysis of two flume structures.
- Location and type of canal crossings, including bridges and pipelines.
- Location and type of turnouts, of either side canals or pipelines.
- Location and type of nearby facilities that could be exposed to flooding or excessive waterlogging should the earthquake damage the canal.
- Hydraulic capacity and required flows of the canal. While this report provides no guidance as to how to calculate these values, the assessment of whether a canal is in minor, moderate or major damage states may depend upon how much loss of flow capacity is tolerable, and for what duration in time following the earthquake.

7.1 Factors that Cause Damage to Canals

A set of performance goals is suggested to describe the performance of a canal in an earthquake. The ideal performance is "no damage." Given that hydraulic performance of a canal is of key importance, the following descriptions define the damage states for canals under seismic loading:

- **No damage.** The canal has the same hydraulic performance after the earthquake.
- **Minor damage.** Some increase in the leak rate of the canal has occurred. Damage to the canal liner may occur, causing increased friction between the water and the liner and lowering hydraulic capacity. The liner damage may be due to PGDs in the form of settlements or lateral spreads due to liquefaction, movement due to landslide, offset movement due to fault offset, or excessive ground shaking. Landslide debris may have entered into the canal causing higher sediment transport, which could cause scour of the liner or earthen embankments. Overall, the canal can be operated at up to 90% of capacity without having to be shut down for make repairs.
- **Moderate damage.** Some increase in the leak rate of the canal has occurred. Damage to the canal liner has occurred, causing increased friction between water and the liner, lowering hydraulic capacity. The liner damage may be due to PGDs in the form of settlements or lateral spreads due to liquefaction, movement due to landslide, offset movement due to fault offset, or excessive ground shaking. Landslide debris may have entered into the canal causes higher sediment transport, which could cause scour of the liner or earthen embankments. Overall, the canal can be operated in the short term at up to 50% to 90% of capacity; however, a shutdown of the canal soon after the earthquake will be required to make repairs. Damage to canal overcrossings may have occurred, and temporary shutdown of the canal is needed to make repairs. Damage to bridge abutments could cause constriction of the canal's cross-section to such an extent that it causes a significant flow restriction.
- **Major damage.** The canal is damaged to such an extent that immediate shutdown is required. The damage may be due to PGDs in the form of settlements or lateral spreads due to liquefaction, movement due to landslide, offset movement due to fault offset, or excessive ground shaking. Landslide debris may have entered the canal and caused excessive sediment transport, or may block the canal's cross-section to such a degree that the flow of water is disrupted, overflowing over the canal's banks and causing subsequent flooding. Damage to overcrossings may have occurred, requiring immediate shutdown of the canal. Overcrossing damage could include the collapse of highway bridges and leakage of non-potable material pipelines such as oil, gas, etc.. Damage to bridge abutments could cause constriction of the canal's cross-section to such an extent that a significant flow restriction which warrants immediate shutdown and repair.

7.2 Vulnerability Assessment of Canals

A vulnerability assessment of canals can be done as follows:

- Establish a spreadsheet which lists the canal reaches at various mileposts where the seismic hazard or the canal design changes.
- Calculate the potential for each of the four seismic hazards for each section of the canal. For liquefaction and landslide hazards, the native soils beneath and nearby the canal should be considered, as well as the soil materials that form the embankments of the canal. For in-line tunnels, the hazards include landslide and tunnel portals which can either affect the tunnel or deposit debris into the canal.
- Estimate the potential for cracking of the liner due to ground shaking hazard. The strain in the liner can be estimated using strain = V/c (where V = peak ground velocity, c = wave propagation speed) type calculations, with attendant estimate of crack size and spacing. Assess if the cracking is due to ground shaking and whether it places the canal in a damage state.
- Estimate the damage state for each length of the canal based on all four seismic hazards.

It is beyond the scope of this report to establish models that can be used for geotechnical assessment of canal embankments. Based on the limited empirical evidence, the following rough guidelines might be useful when detailed geotechnical assessments are lacking:

- Minor damage to unreinforced liners or unlined embankments may be expected at a rate of 0.1 repairs per kilometer for ground shaking velocities of PGV = 20 to 35 inches per second. The minor damage rate drops to 0.01 repairs per kilometer for ground shaking velocities of PGV = 5 to 15 inches per second, and 0 below that. Damage to reinforced liners is one quarter of these rates. Bounds on the damage estimate can be estimated assuming plus 100% to minus 50% at the plus or minus one standard deviation level, respectively.
- Moderate damage is expected if lateral or vertical movements of the embankments due to liquefaction or landslide are in the range of 1 to 5 inches. Moderate damage occurs due to fault offset across the canal of 1 to 5 inches. Moderate damage is expected if small debris flows into the canal from adjacent landslides.
- Major damage is expected if PGDs of the embankments are predicted to be six inches or greater. Major damage occurs due to fault offset across the canal of six inches or more. Major damage is expected if a significant amount of debris is predicted to flow into the canal from adjacent landslides. The differentiation of moderate or major damage states for debris flows into the canal should factor in hydraulic constraints caused by the size of the debris flow, the potential for scour due to the type of debris and water quality requirements.

7.3 References

Bureau of Reclamation, "Linings for irrigation canals," 1963.

Knarr. M., "Seismic Modification of Open Flume Structures on the Borel Canal near Lake Isabella, California," in *Seismic Evaluation and Upgrade of Water Transmission Facilities*, Eds. J. Eidinger and E. Avila, TCLEE Monograph No. 15, ASCE, 1999.

8.0 In-line Components

Various types of in-line components exist along water transmission pipelines, including portions of the supervisory control and data acquisition (SCADA) system located along the conveyance system and various flow control mechanisms (e.g., valves and gates). This section highlights the main seismic vulnerabilities for these in-line components.

8.1 Pipeline Valves

The fragility information presented in Section 4 includes damage to in-line valves along pipeline systems.

Most pipeline valves are buried in the soil along with the pipe. The valves can be of many types, including gate valves, butterfly valves, ball valves, check valves, etc. The fragility algorithms presented in Section 4 include damage that might occur to these valves. Some data is presented in Appendix A with consideration for the breakdown of pipeline damage data in terms of damage to pipe joints, pipe body and appurtenances such as valves; this could be used as a first order estimate for damage to valves.

In a few cases for larger diameter pipes, pipeline valves will be located in buried concrete vaults. Normally, the length of pipe in the buried vault is only 4 to 5 pipeline diameters, and amplified inertial response of the above-ground pipe-valve-pipe system within the vault is not significant. However, in cases where there are long runs of pipe, such that the pipe-valve frequency is much less than about 10 hertz, the potential for increased stresses in the pipeline exists, along with an increased chance of damage. For these cases, it is reasonable to evaluate the pipe-valve system using code-based rules such as those provided in the ASME B31.1 code. When performing such analyses, care should be taken to account for relative stiffness issues at large-pipe-to-small-pipe connections, where pipes enter or leave the concrete vault, and at pipe support locations. These are the areas that may be most prone to damage.

8.2 SCADA Equipment

In-line SCADA hardware includes a variety of components, including:

- Instrumentation
- Power Supply (normal, backup)
- Communication components (normal, backup)
- Weather enclosures (electrical cabinets and vaults)

Many modern SCADA instruments use solid state equipment. The sensor equipment is attached to the pipeline and the signal processing equipment is located in a metal cabinet enclosure. The dominant vulnerabilities for this equipment are batteries falling over, circuit boards dislodging and gross movement of the cabinet enclosure because of inadequate anchorage. The best way to discover these vulnerabilities is by a site-specific inspection.

Some SCADA equipment installations include instruments that measure pressure or flow based on the height of water in a tube. During earthquake conditions, hydraulic transients can introduce air into the pipeline. These hydraulic transients arise from the pipe failure or inertial response of pipes. Once air is introduced into the pipes, it can reach the instrument location and cause the instruments to provide incorrect readings. While the instrument is not damaged, it will require recalibration after the earthquake.

Communication between the in-line equipment and the central SCADA computers is by one of two methods: landline telephone or radio.

- Landline telephone wires are usually seismically rugged and, in some cases, may traverse areas prone to large PGDs. Telephone wires are capable of withstanding large PGDs (often several feet) before they become non-functional. Once the telephone wire reaches the telephone company central office, the signal is usually routed directly to the centralized SCADA computer location. As long as the central office is functional, the signal will reach the SCADA computer location. However, in some cases, the landline may be on a “switched” network, and because of telephone system saturation for the first few days after an earthquake, the signal may be disrupted.

- Radio communication networks can be disrupted by earthquakes. The radio link from the in-line component to the central SCADA computer may have to be transmitted via repeater stations on hilltops or building roofs. All equipment that supports the radio link must be seismically rugged, have adequate backup power and be located in buildings that will not suffer heavy damage. These vulnerabilities are best verified by field investigation.

8.3 Canal Gate Structures

Some damage to in-line gate structures in canals has occurred in past earthquakes. The small amount of empirical information for gate structures and the wide variation in possible arrangement of gate structures precludes the development of gate-specific fragility curves. Seismic evaluation of gate structures should consider all the seismic hazards, as well as fluid-imposed forces.