

#7189546 Overview of Engineering Options for Increasing Infrastructure Resilience

# INCREASING INFRASTRUCTURE RESILIENCE BACKGROUND REPORT

February 2019

miyamoto.

WORLD BANK GROUP

**About the Project**

The Overview of Engineering Options for Increasing Infrastructure Resilience Project, Contract #7189546, is funded by the World Bank Group with the Global Facility for Disaster Reduction and Recovery. The objective of this project is to prepare a flagship report about infrastructure resilience that investigates the impacts of natural disasters from the loss of (lifeline) infrastructure services and from supply-chain effects. This project also aims to support the development of public policies and interventions that make economic systems more resilient.

**About the World Bank Group**

The World Bank Group is one of the world's largest sources of funding and knowledge for developing countries. Its five institutions share a commitment to reducing poverty, increasing shared prosperity, and promoting sustainable development. The World Bank Group is committed to open development and has opened its data, knowledge, and research to foster innovation and increase transparency in development.

**About the Global Facility for Disaster Reduction and Recovery**

The Global Facility for Disaster Reduction and Recovery (GFDRR) is a global partnership that helps developing countries better understand and reduce their vulnerability to natural hazards and climate change. Launched in 2006, GFDRR provides technical and financial assistance to help disaster-prone countries decrease their vulnerability and adapt to climate change.

**Miyamoto International**

Miyamoto International is a global earthquake + structural engineering, project management, and construction management company that provides critical services that sustain industries and safeguard communities around the world. Miyamoto specializes in Disaster Resiliency Engineering along with disaster risk mitigation, response, and reconstruction.

**Disclaimer**

The opinions, findings, and conclusions stated herein are those of the authors and do not necessarily reflect the views of the World Bank Group or GFDRR.

**Acknowledgment**

The data and work presented in this document are based on technical research and data presented by a number of authors. When available, these contributions are listed as part of the references. The pictures, photographs, and other graphical information that are used in this report are based on the contributions of many organizations and individuals and were obtained from the internet.

Submitted on:

February 27, 2019

By:

Miyamoto International, Inc.

www.miyamotointernational.com

## SYNOPSIS

As shown in Figure 1, many parts of the world are subject to a variety of natural hazards. As the population of the world has increased, people live in and infrastructure has been built in locations where natural hazard impacts are severe. Critical infrastructure (power, transport, and water assets) are particularly vulnerable to natural hazards. Damage to these components has a cascading impact that extends not only to the assets themselves, but also to the population at large and to local and national economies. Accordingly, improvements in design and construction that can reduce the vulnerability and that are cost-effective can enhance the resilience of surrounding communities.

The main report for this World Bank Group–sponsored project summarizes the infrastructure that was considered, the expected level of damage, and the suggested improvements, and it provides an estimate of the costs and benefits that are associated with such improvements. This background document presents a more detailed treatment of the topic and provides background information and supporting data.

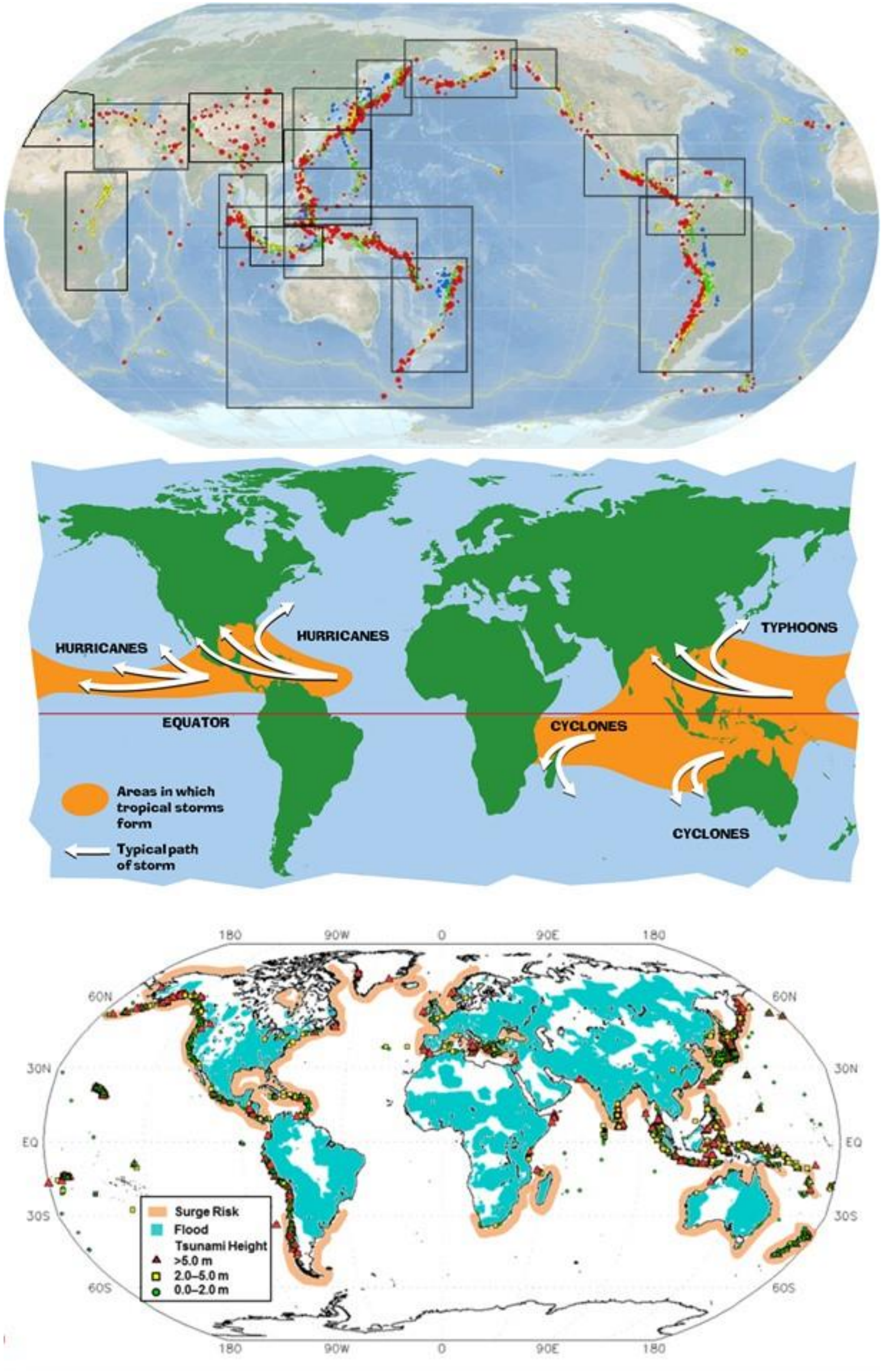



Figure 1. Top to bottom: global earthquake, wind (hurricane shown), and flood hazard maps

**Units of measurement**

This report is based on the review of a number of references. Whereas some of the references provide SI (metric) units, others use English units. Therefore, both metric and English units are used in this report. For reference, following are common conversions:

- Distances
  - 1 foot = 0.305 m
  - 1 inch = 25.4 mm
  - 1 mile = 1.61 km
- Velocity
  - 1 mph (miles per hour) = 1.61 km/h = 0.45 m/sec.
- Flow
  - 1 cfs (ft.$^3$/sec) = 0.0283 m$^3$/sec.
- Pressure
  - 1 ksi (kip/in.$^2$) = 6.89 MPa

## CONTENTS

## List of Tables

## LIST OF FIGURES

# 1. Subject matter

## 1.1 Infrastructure assets

### *1.1.1 Power*

- Thermal power plants (coal, gas, or oil)
- Hydropower plants
- Solar farms
- Wind farms
- Nuclear power plants
- Substations
- Transmission and distribution systems

### *1.1.2 Transport*

- Railways (diesel and electric)

### *1.1.3 Water*

- Reservoirs (open and storage tanks)
- Water treatment plants (potable water and wastewater)
- Distribution pipes
- Sewage network emissaries
- Water conveyance systems (canals)
- Drainage systems

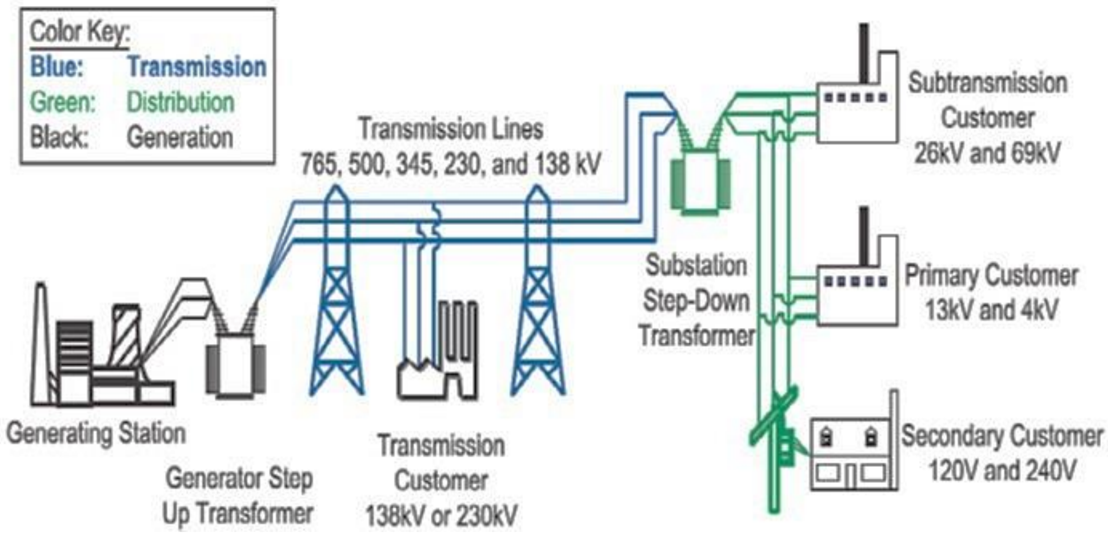
## 1.2 Roadways (on grade and bridges)

- Highways on grade
- Highway bridges
- Paved secondary roads on grade
- Secondary urban road bridges
- Unpaved tertiary roads
- Wooden bridges

## 1.3 Hazards

- Earthquakes ground motion
- Liquefaction
- Wind
- Flood
- Landslide (roadways only)

## 2. POWER INFRASTRUCTURE



## 2.1 Thermal power plants (coal, gas, or oil)

### *2.1.1 Overview*

A thermal power station is an asset in which heat energy (from coal, gas, or oil) is converted to electric power. In most places throughout the world, the turbine is steam-driven. Water is heated, turns into steam, and spins a steam turbine, which drives an electrical generator. Figure 2 shows an example of a thermal power plant.

Figure 2. Thermal power plant

### *2.1.2 Summary*

The results from a literature review of thermal power plants are summarized in Table 1 and are presented in the "*Improvement of infrastructure resilience evaluation matrix for selected natural hazards*" table of the project final report. The following subsections of this background report provide more detailed descriptions and background information*.

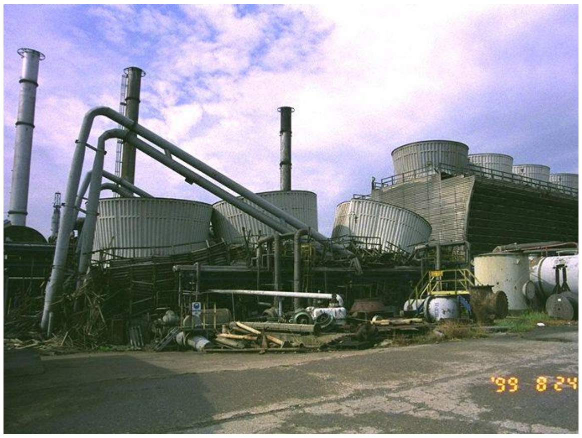
| Hazard | Susceptible | Improvements | Damage index | | Approximate improvement cost as % of cost |
|---|---|---|---|---|---|
| | | | As-is | Improved | |
| **Earthquake** | Y | Anchorage, seismic components | 0.25 | 0.02 | 20% |
| **Liquefaction** | Y | Deep foundation | 0.3 | Low | 20% |
| **Wind** | Y | Higher factor of safety | 0.4 | 0.1 | 10% |
| **Flood** | Y | Elevated, use flood wall or sheet piling | 0.05 | Low | 2% |

Table 1. Summary of findings for thermal power plants

### *2.1.3 Vulnerability to natural hazards*

Thermal power plants have been significantly damaged in past natural hazard events. Figure 3 presents examples.

* % of cost denotes replacement cost (similar to the total cost) in this and subsequent similar tables in the Summary section of all infrastructure types.



Earthquake (Izmit, Turkey, Earthquake, 1999)

Liquefaction (Hokkaido Earthquake, Japan, 2018)

Wind (Hurricane Sandy, 2012)

Flood (aftermath of Hurricane Florence, 2018)

Figure 3. Damage to thermal power plants from natural hazards

### *2.1.4 Earthquake hazard*

#### 2.1.4.1 General

Thermal power plants are susceptible to damage from peak ground acceleration (PGA). Damage states are defined as listed in Table 2 (FEMA 2013a).

| Damage state | Definition | Status | Power |
|---|---|---|---|
| DS0 (none) | -- | Operational | Normal |
| DS1 (minor) | Turbine tripping, or light damage to diesel generator, or the building being in minor damage state | Operational | Close to nominal |
| DS2 (moderate) | Chattering of instrument panels and racks, considerable damage to boilers and pressure vessels, or the building being in moderate damage state | Operational without repair | Reduced |
| DS3 (extensive) | Considerable damage to motor driven pumps, or considerable damage to large vertical pumps, or the building being in extensive damage state | Operational after repair | No |
| DS4 (complete) | Extensive damage to large horizontal vessels beyond repair, extensive damage to large motor operated valves, or the building being in complete damage state | Not repairable | No |

Table 2. Earthquake damage states for power-generation plants

### 2.1.4.2 Key metric for consideration

A key consideration for thermal power plants is the amount of time that it takes to restore operations after an earthquake. Figure 4 shows the timeline of restoration (FEMA 2013a) for thermal power plants.

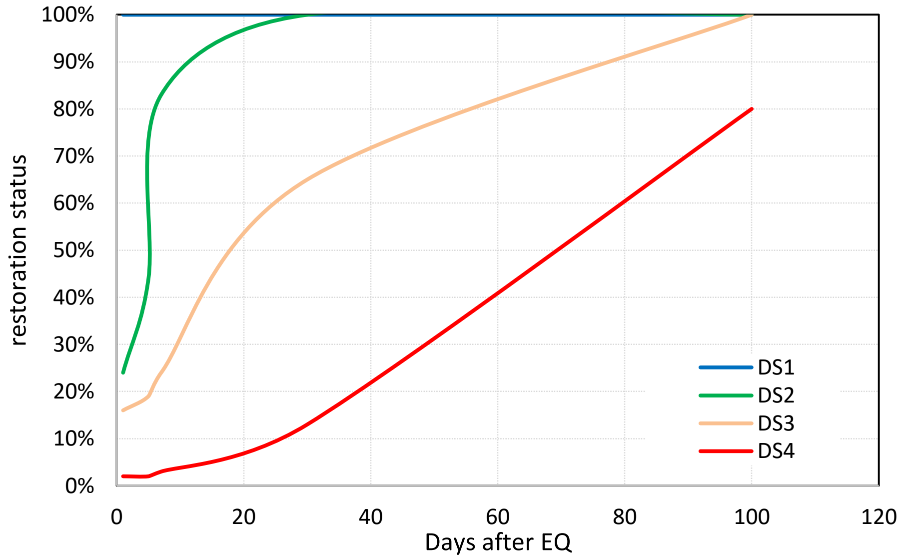



Figure 4. Restoration curve, power-generation plants, earthquake hazard

### 2.1.4.3 Infrastructure improvements

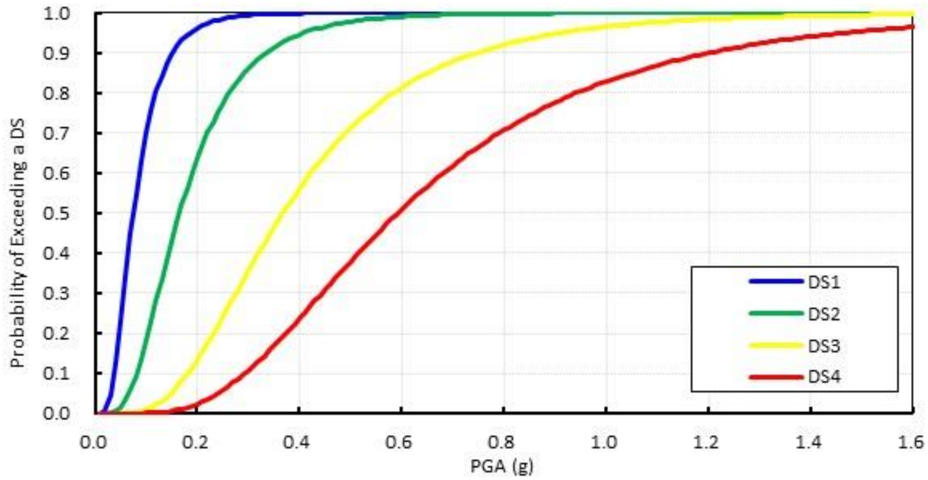
For thermal power plants, earthquake performance can be improved by several methods. Examples include:

- Ensure that all buildings meet the current seismic code requirements and retrofit the buildings as needed.
- Provide proper anchorage of all components, including boilers, tanks, etc.
- Use only tested or analyzed components that have shown robustness during evaluation.
- Use seismic protection devices to reduce demand on the components or on the buildings.
- Ensure that a seismic switch is installed to allow safe shutdown and safe restart of the plant.
- Perform routine and regular maintenance for the facility and fix any observed problems.

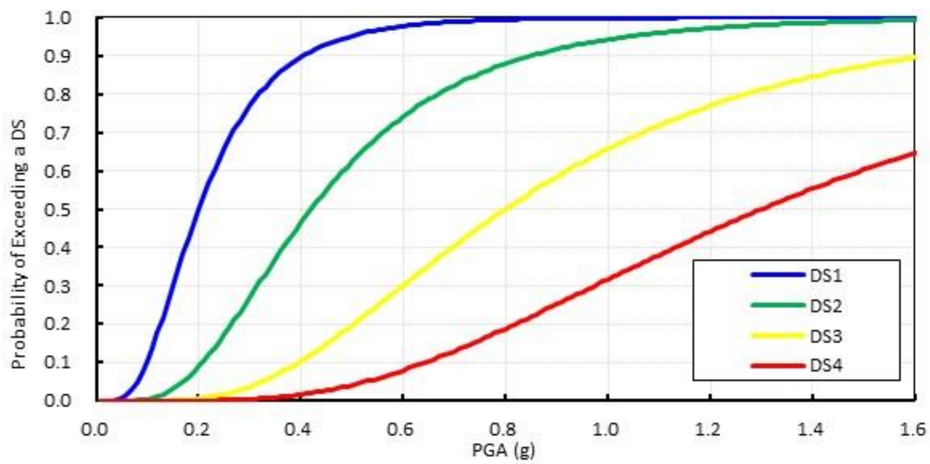
### 2.1.4.4 Cost-benefit considerations

FEMA (2013a) provides fragility functions for power plants in the United States. Two sets of data are provided: one for power-generation plants that have standard components and one for plants that have proper anchorage and seismic components and for which the building design meets the requirements of modern seismic codes. In this report, the fragility parameters are modified for the unanchored case to account for the higher variability and the lower expected quality in worldwide application, and for the more robust seismic design of buildings and components, the seismic-resilient parameters are increased.

Figure 5 presents the fragility functions for both non-seismic and enhanced power plants, respectively. Note that for an earthquake with a PGA of 0.4g (a value that could be expected from a $M_w$ 7 earthquake), the probability of exceeding DS4 is 22% for the non-seismic power stations and is 2% for the seismic power stations. In other words, by using seismic components (albeit at a slightly higher cost, estimated at approximately 20% in this report), the probability of full power restoration and supply is increased from approximately 65% to 95% in 1 month (30 days). Such expedited restoration provides significant economic benefits and helps in the community recovery.




Unanchored subcomponents




Anchored and seismic subcomponents

Figure 5. Damage fragility functions for power plants

### *2.1.5 Liquefaction hazard*

#### 2.1.5.1 General

Thermal power plants that are constructed on weak soils in earthquake zones are vulnerable to damage from liquefaction.

#### 2.1.5.2 Key metric for consideration

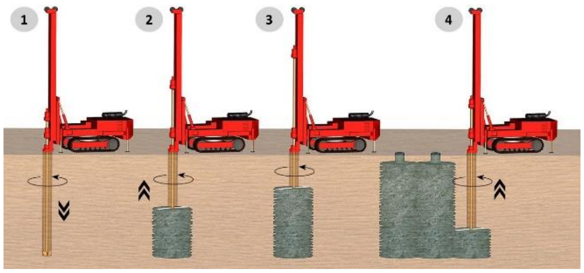
Structural damage is used to assess the vulnerability of thermal power plant components. Liquefaction can damage thermal power plants and result in loss of operation. Because the failure is below ground, the cost of geotechnical repair and restoration of operations is high.

#### 2.1.5.3 Infrastructure improvements

For thermal power plants, liquefaction performance can be improved by several methods. Examples include:

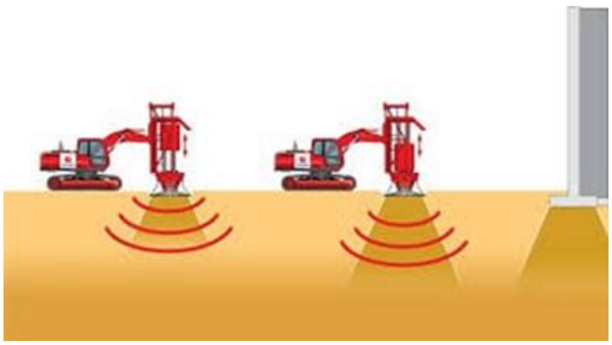
- Use soil improvements; see Figure 6.

- Use a deep foundation; see Figure 7.

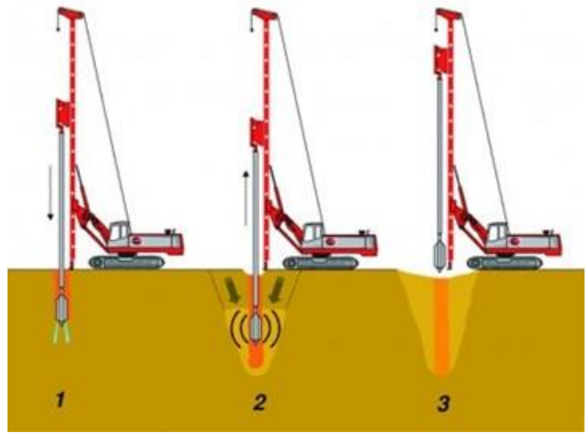



Jet grouting

Earthquake drains



Rapid impact compaction




Vibro compaction

Figure 6. Example of geotechnical liquefaction mitigation

Steel driven piles

Micropiles

Concrete driven piles

Concrete drilled piles

Figure 7. Example of deep foundations

#### 2.1.5.4 Cost-benefit considerations

For thermal power plants that are constructed on a shallow foundation, liquefaction can result in damage and in loss of operation and power for an extended period. Power restoration would be costly and long delayed. By contrast, thermal power plants that are constructed by using deep foundations (piles) driven past the liquefiable layers or that are constructed in conjunction with soil mitigation would be immune to such extensive damage. Assuming that the substructure cost is 20% of the total thermal power plant cost, a deep foundation could add approximately 20% to the construction cost. However, this foundation cost is significantly lower than the long-term loss of revenue that liquefaction damage can cause to thermal power plants.

### *2.1.6 Wind hazard*

#### 2.1.6.1 General

The tall components (such as cooling towers or flue-gas stacks) of thermal power plants are susceptible to damage from wind. Such damage can result in loss of operation of the thermal power plant. The wind damage could be due to direct wind or crosswind (such as vortex shedding).

#### 2.1.6.2 Key metric for consideration

Structural reliability is used to assess the vulnerability of thermal power plant components. Structural reliability is the probability that the structure will not reach a limit state (e.g., a failure state) during a given period of strong winds. With this approach, the responses of various thermal power plant components are averaged out, thus it eliminates some of the uncertainties.

The probability of failure is related to the factor of safety that is used in the design; a higher factor of safety implies a lower probability of failure from wind events.

#### 2.1.6.3 Infrastructure improvements

For thermal power plants, wind performance can be improved by several methods. Examples include:

- Design the thermal power plant buildings, stacks, and tower components for a higher design wind speed; see Figure 8.

- Ensure that all components and buildings are properly anchored by using positive mechanical attachments that are wind-rated.
- Positively anchor all building roofs to columns and walls.
- Brace cooling towers.
- Provide crosswind mitigation for stacks.

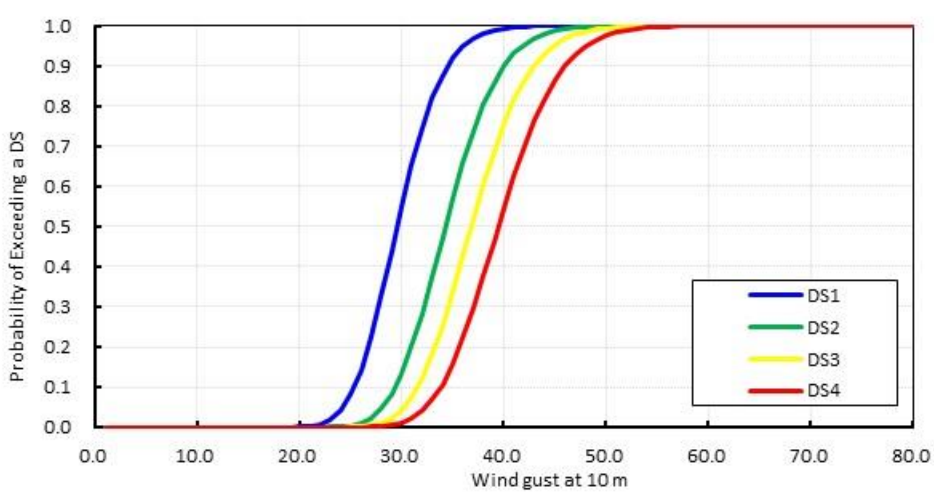
Braced cooling tower support structure

Helical strake for crosswind

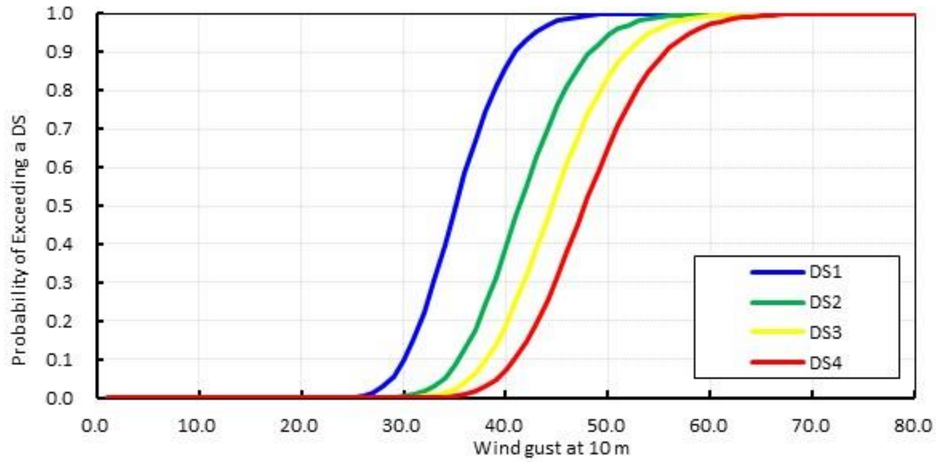
Figure 8. Examples of wind mitigation for thermal power plants

### 2.1.6.4 Cost-benefit considerations

Figure 9 presents the fragility functions for minimum wind design and enhanced wind design (adopted from Ham et al. 2009), respectively. Note that for wind gusts of 40 m/sec, the probability of a complete damage state (DS4) is reduced from approximately 40% to 10% with mitigation measures. In other words, by using proper wind design and wind-resistant attachment components (albeit at a slightly higher cost, estimated at approximately 10% in this report), the probability of damage is significantly reduced.




Minimum wind design (equivalent to exposure B)




Enhanced wind design (equivalent to exposure D)

Figure 9. Damage fragility functions for power plants

Thermal power plants that are designed per code, with no factor of safety, have a 20% chance of power plant failure when a large windstorm occurs. This probability of failure is reduced by a factor of nearly 2 when a more robust design (at a slightly higher cost) is implemented.

### *2.1.7 Flood hazard*

#### 2.1.7.1 General

Thermal power plants are susceptible to damage from flooding. Flooding can damage expensive components or other components that, when damaged, lead to extended loss of power. Thermal power plants are particularly vulnerable to flooding. Vulnerabilities are defined as listed in Table 3 (FEMA 2013c).

| **Overall vulnerability** | **Inundation** | **Scour** | **Hydraulic pressure & debris** | **Financial loss** | **Loss of operation** |
|---|---|---|---|---|---|
| High | High | -- | -- | High | High |

Table 3. Flood vulnerability for power-generation plants

#### 2.1.7.2 Key metric for consideration

The status of damage (FEMA 2013c) for power plants is presented in Figure 10. At the onset of flooding, support facilities are damaged at ground level. Control and generation facilities are damaged when water elevation reaches the second level.

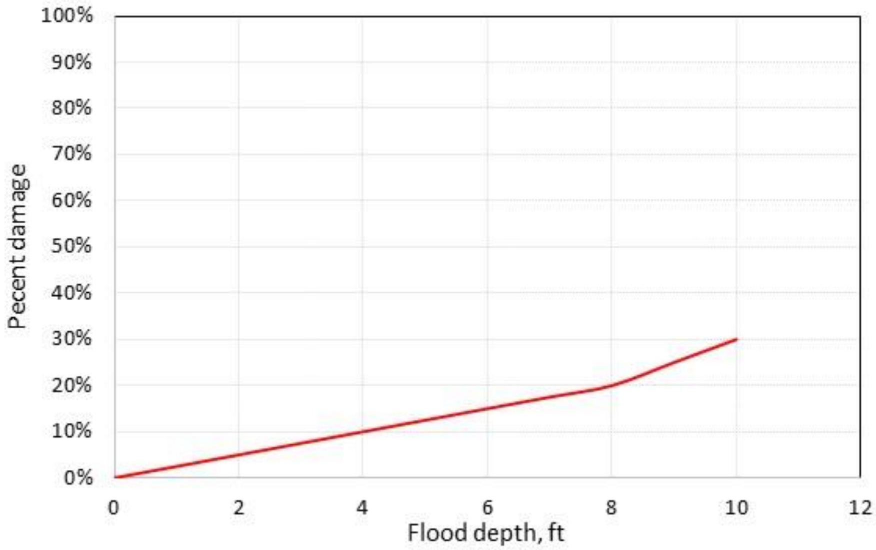



Figure 10. Percentage of damage as a function of inundation depth, power plants

#### 2.1.7.3 Infrastructure improvements

For thermal power plants, flood performance can be improved by several methods. Examples include:

- Install flood-monitoring devices to notify operators when flooding occurs and as it reaches certain levels.
- Provide dikes that are designed for a 500-year flood as protection to minimize overtopping.
- Base the design for the control building and use that floor elevation (if several buildings have different floor slab elevations).
- Provide flood protection walls; see Figure 11.

Flood wall Steel sheet piling

Figure 11. Examples of flood mitigation for thermal power plants

#### 2.1.7.4 Cost-benefit considerations

FEMA (2013c) provides flood damage data for power plants in use in the United States. The expected damage is typically correlated to the inundation depth as shown in Figure 10. For thermal power stations without protection, flood inundation of 2 ft. implies damage of 5%. For typical thermal power plants, this level of damage could be approximately 5% of the thermal power plant cost (US $25 million). The cost of constructing steel sheet piles is estimated at approximately 2% of the thermal power plant cost.

## 2.2 Hydropower plants

### *2.2.1 Overview*

A hydropower plant is a power station in which heat energy from fast-falling water is converted to electric power. In most places throughout the world, this power comes from the potential energy of dammed water that drives a water turbine and a generator. The power that is extracted from the water depends on the volume of water and on the difference in height between the source and the water's outflow. A large pipe (the "penstock") delivers water from the reservoir to the turbine. Figure 12 shows an example of a hydropower plant.

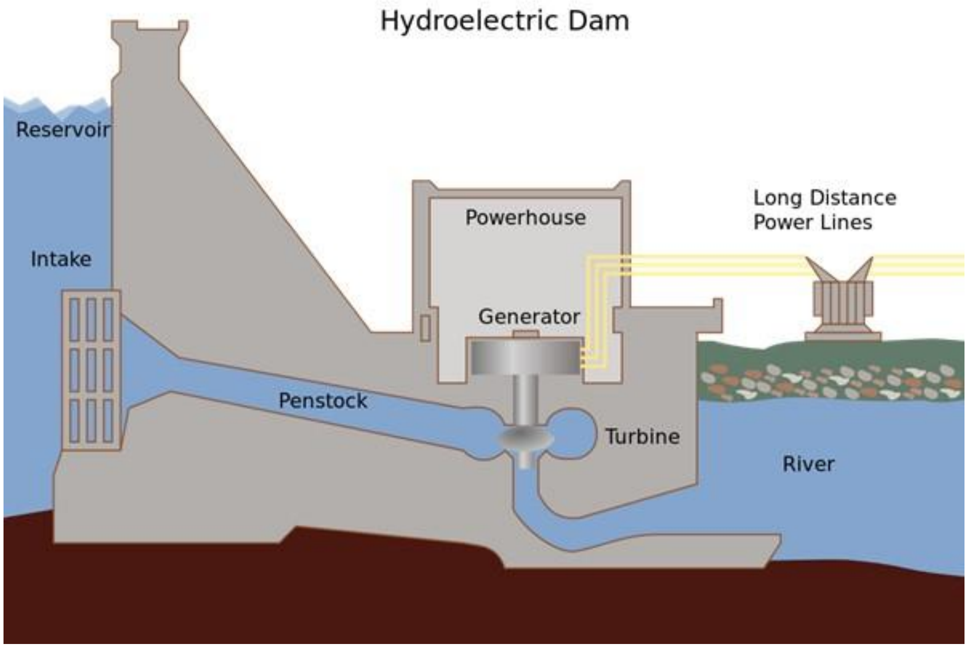



Figure 12. Hydropower plant

### *2.2.2 Summary*

The results from a literature review of hydropower plants are summarized in Table 4 and are presented in the "*Improvement of infrastructure resilience evaluation matrix for selected natural hazards*" table of the final report. The following subsections of this background report provide more detailed descriptions and background information.

| Hazard | Susceptible | Improvements | Damage index | | Approximate improvement cost as % of cost |
|---|---|---|---|---|---|
| | | | As-is | Improved | |
| **Earthquake** | Y | Higher threshold seismic design | 0.7 | 0.4 | 20% |
| **Liquefaction** | N | -- | -- | -- | -- |
| **Wind** | N | -- | -- | -- | -- |
| **Flood** | Y | Enhanced spillway design | 0.1 | 0.05 | 3% |

Table 4. Summary of findings for hydropower plants

### *2.2.3 Vulnerability to natural hazards*

Hydropower plants have been significantly damaged in past natural hazard events. Figure 13 presents examples.

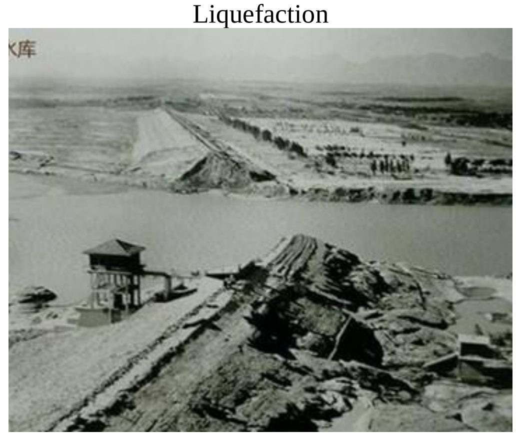
Earthquake (Sichuan, China, 2008 Earthquake)

This hazard does not apply.

Liquefaction

This hazard does not apply.

Wind



Flood (Banqiao Reservoir Dam, 1975 flood)

Figure 13. Damage to hydropower plants from natural hazards

### *2.2.4 Earthquake hazard*

#### 2.2.4.1 General

Hydropower plants are susceptible to damage from peak ground acceleration (PGA). Modes of failure could include:

- Sliding or lifting (overturning) at the base of the dam
- Sliding at lift joints
- Structural failure of bottom outlets, spillways, intake towers, or gates
- Failure of electrical or mechanical equipment or its anchorage

Damage states are defined as listed in Table 5 (FEMA 2013a).

| Damage state | Definition | Status | Power |
|---|---|---|---|
| DS0 (none) | -- | Operational | Normal |
| DS1 (minor) | Turbine tripping, or light damage to diesel generator, or the building being in minor damage state | Operational | Close to nominal |
| DS2 (moderate) | Chattering of instrument panels and racks, considerable damage to boilers and pressure vessels, or the building being in moderate damage state | Operational without repair | Reduced |
| DS3 (extensive) | Considerable damage to motor driven pumps, or considerable damage to large vertical pumps, or the building being in extensive damage state | Operational after repair | No |
| DS4 (complete) | Extensive damage to large horizontal vessels beyond repair, extensive damage to large motor operated valves, or the building being in complete damage state | Not repairable | No |

Table 5. Earthquake damage states for hydropower plants

### 2.2.4.2 Key metric for consideration

A key impact metric for hydropower plants is the amount of time that it takes to restore operations. Figure 14 shows the timeline of restoration (FEMA 2013a) for hydropower plants.

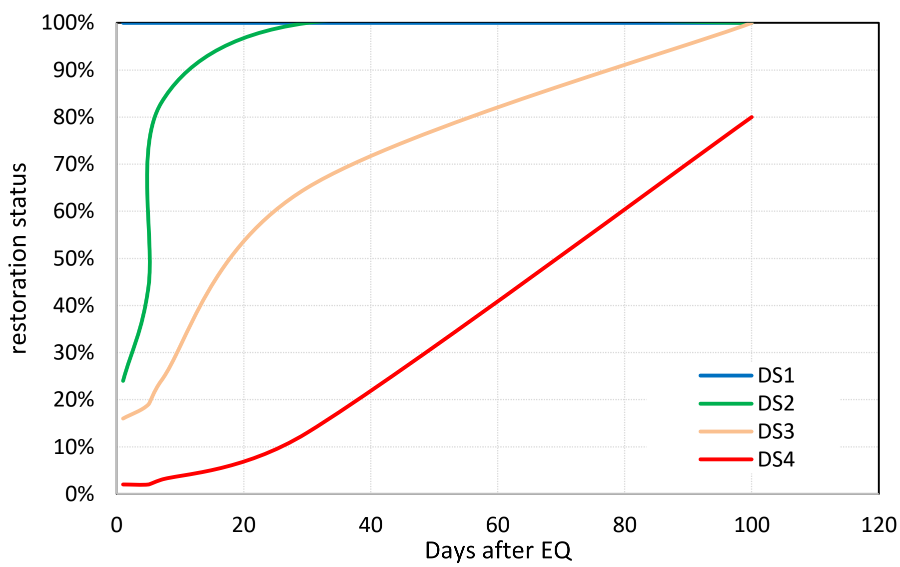



Figure 14. Restoration curve, hydropower plants, earthquake hazard

### 2.2.4.3 Infrastructure improvements

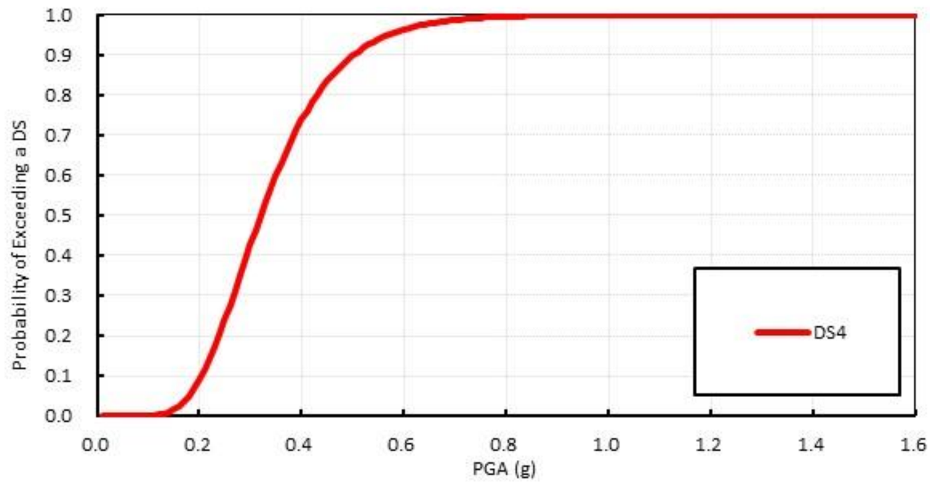
For hydropower plants, earthquake performance can be improved by several methods. Examples include:

- Design spillway gates, other gates, and valves at the bottom outlet for at least a Safety Evaluation Earthquake (SEE) or a 2,500-year event.
- Ensure that all electrical and mechanical components are designed and installed per seismic requirements.
- Account for hydrodynamic pressure in penstocks and tunnels.

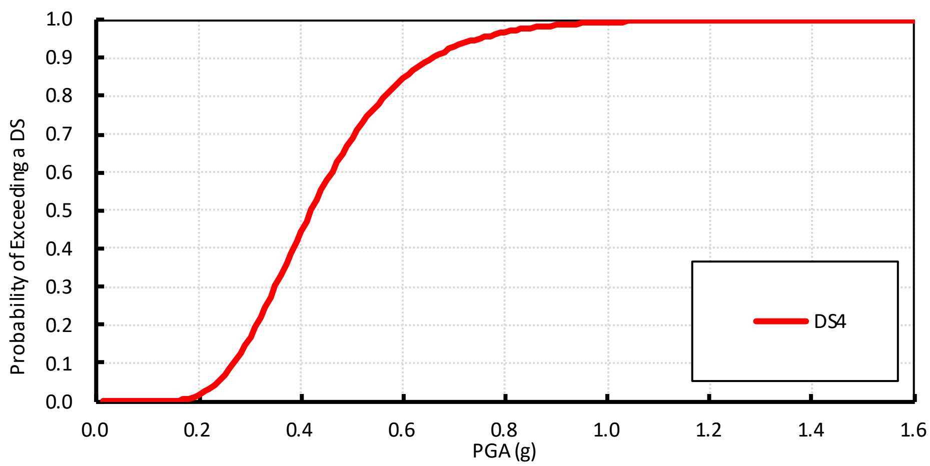
- Ensure that a seismic switch is installed to allow safe shutdown and safe restart of the plant.
- Perform routine and regular maintenance for the facility and fix any observed problems.

#### 2.2.4.4 Cost-benefit considerations

Ghanaat et al. (2012) studied concrete dams and developed fragility plots for several modes of failure. They examined the base-sliding mode of failure as one of the damage states. Their analysis was based on a case for a well-constructed unit. In this report, that case is used as a seismic case, and a non-seismic case is implied by using 75% of the median value. Figure 15 presents the fragility functions for the two cases. Note that for an earthquake with a PGA of 0.4g, the probability of exceeding DS4 from base sliding is 70% for the non-seismic case and is 40% for the seismic case. In other words, by using seismic design and detailing (estimated at approximately 20% of total cost in this report), the probability of sliding at the base is reduced by a factor of close to 2.




Unanchored subcomponents




Anchored and seismic subcomponents

Figure 15. Damage fragility functions for hydropower plants

### *2.2.5 Liquefaction hazard*

#### 2.2.5.1 General

Hydroelectric power plants are constructed in areas that have fast-falling water. This feature typically implies a hard rock subbase and, as such, liquefaction is not considered to be a problem.

#### 2.2.5.2 Key metric for consideration

This topic does not apply.

#### 2.2.5.3 Infrastructure improvements

This topic does not apply.

#### 2.2.5.4 Cost-benefit considerations

This topic does not apply.

### *2.2.6 Wind hazard*

#### 2.2.6.1 General

Massive hydroelectric power plants are not adversely affected by windstorms and, being typically located away from shorelines, they are not affected by hurricanes. Buildings, power lines, towers, and other components that hydroelectric power plants depend on could be vulnerable to wind damage and are addressed in other sections of this report.

#### 2.2.6.2 Key metric for consideration

This topic does not apply.

#### 2.2.6.3 Infrastructure improvements

This topic does not apply.

#### 2.2.6.4 Cost-benefit considerations

This topic does not apply.

### *2.2.7 Flood hazard*

#### 2.2.7.1 General

Hydropower plants are one means for providing flood protection downstream. However, they themselves can be subject to flooding.

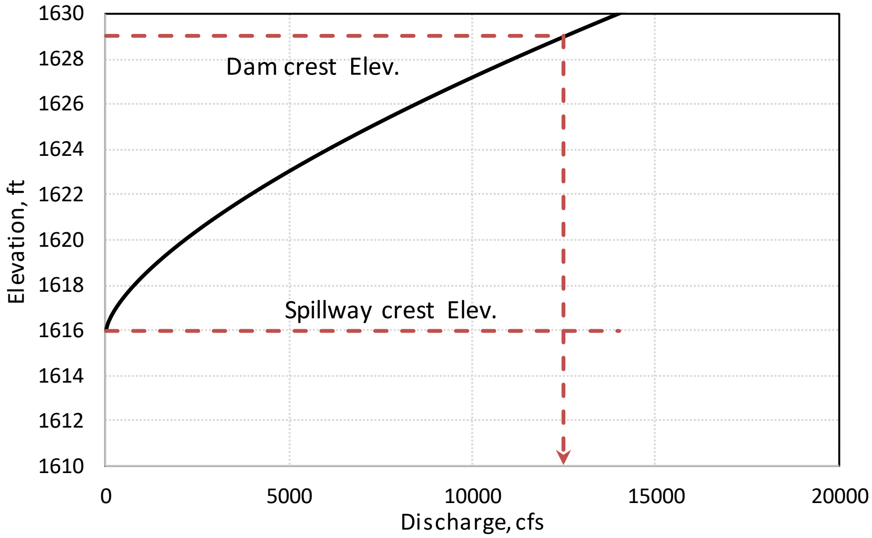
#### 2.2.7.2 Key metric for consideration

The key consideration is to avoid overtopping of the dam during extreme flooding.

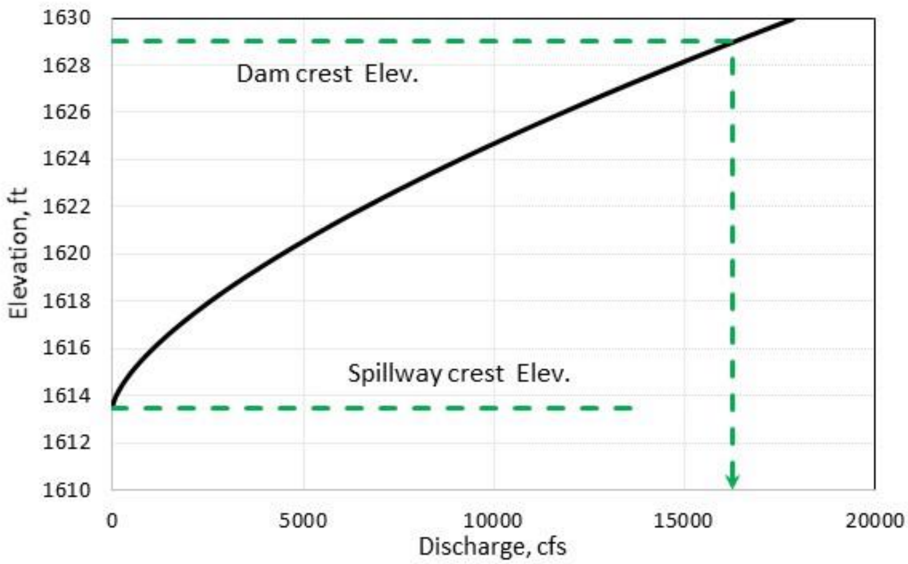
#### 2.2.7.3 Infrastructure improvements

For hydropower plants, flood performance can be improved by several methods. Examples include:

- Ensure that the reservoir is properly drenched regularly to remove excess sediment and to reduce the chance of overtopping.
- Ensure that the dam has adequate spillway capacity and that there is remote access to operate the spillways.
- Increase flow capacity by modifying the spillway elevation (H), width (L), or weir coefficient (C); see Figure 16.




13 ft. of waterhead




15 ft. of waterhead

Figure 16. Spillway rating curve (capacity) as a function of spillway elevation

### 2.2.7.4 Cost-benefit considerations

By raising the elevation of a spillway by 2 ft., the flow capacity increases from 12,000 to 16,000 cfs (from 340 to 450 $m^3$/sec). During design, a higher water flow rate can be assumed to result in improved performance and reduced probability of overtopping during flooding.

The additional cost for such improvements is estimated at 3% of the initial capital costs, and the estimated improvement is from a 10% probability of overtopping for the as-is design to less than a 5% probability of overtopping.

## 2.3 Solar farms

### *2.3.1 Overview*

A photovoltaic power station, also known as a “solar park,” is a large-scale photovoltaic system (PV system) that is designed to supply utility power into the electrical grid. Sometimes these stations are also called “solar farms” or “solar ranches,” especially when they are in agricultural areas. The sun’s energy is converted to electrical energy and is added to the utility grid. Figure 17 shows an example of a solar farm.

Figure 17. Solar farm

### *2.3.2 Summary*

The results from a literature review of solar farms are summarized in Table 6 and are presented in the “*Improvement of infrastructure resilience evaluation matrix for selected natural hazards”* table of the project final report. The following subsections of this background report provide more detailed descriptions and background information.

| Hazard | Susceptible | Improvements | Damage index | | Approximate improvement cost as % of cost |
|---|---|---|---|---|---|
| | | | As-is | Improved | |
| **Earthquake** | Y | Anchorage, seismic components | 0.1 | 0.02 | 5% |
| **Liquefaction** | N | -- | -- | -- | -- |
| **Wind** | Y | Proper wind design, connections | 0.2 | 0.08 | 15% |
| **Flood** | Very low | Check for local scour | -- | -- | -- |

Table 6. Summary of findings for solar farms

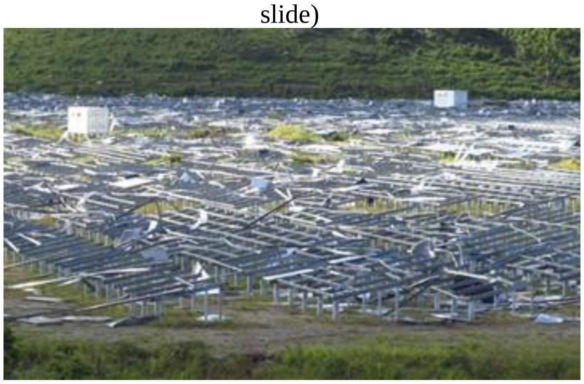
### *2.3.3 Vulnerability to natural hazards*

Solar farms have been moderately damaged in past natural hazard events. Figure 18 presents examples.

Earthquake (Kumamoto Earthquake, foundation slide)

This hazard does not apply.

Liquefaction



Wind (Hurricane Maria, Puerto Rico)

Flood (Flash flooding, United Kingdom)

Figure 18. Damage to solar farms from natural hazards

### *2.3.4 Earthquake hazard*

#### 2.3.4.1 General

Elevated solar farms are light structures and thus are not expected to experience large forces in an earthquake. However, the members and the connections, including the attachment of the PV panels to the support structure, must be designed to meet the demand from seismic loading.

#### 2.3.4.2 Key metric for consideration

Functionality of the system after an earthquake is a key metric for the impact on solar farms.

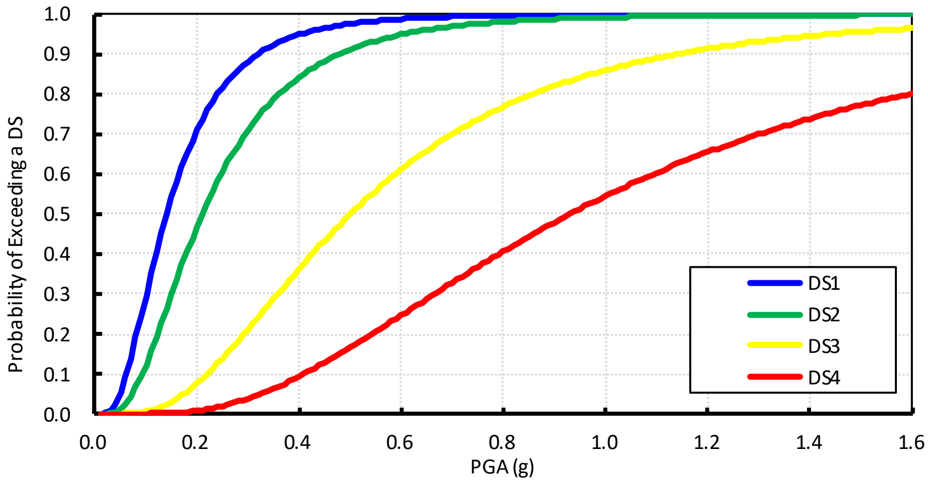
#### 2.3.4.3 Infrastructure improvements

For solar farms, earthquake performance can be improved by several methods. Examples include:

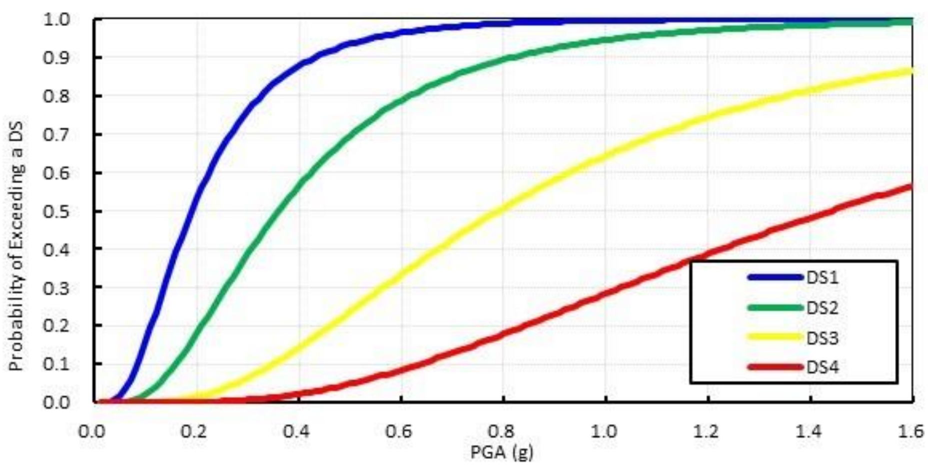
- Seismically design the support structure.
- Ensure that tested or certified connections are used to attach the panels to the support structure.
- Ensure at the baseplates and foundation anchorage are sized and designed properly and check for sliding and overturning.

#### 2.3.4.4 Cost-benefit considerations

FEMA (2013a) provides fragility functions for steel light-frame structures, similar to the solar-panel elevated structures that are used in the United States. Different sets of data are provided depending on the level of compliance with modern seismic codes. In this report, the low and high compliance levels are used to represent typical and enhanced constructions, respectively. Figure 19 presents the fragility functions for both non-seismic and enhanced substations. Note that for an earthquake with a PGA of 0.4g, the probability of exceeding DS4 is 10% for the non-seismic construction and is 2% for the seismic construction. In other words, by using seismic components (at a slightly higher cost, estimated at approximately 5% in this report), the probability of failure is reduced significantly.




Typical construction




Enhanced construction

Figure 19. Damage fragility functions for support frames of solar panels

### *2.3.5 Liquefaction hazard*

#### 2.3.5.1 General

Elevated solar farms are typically constructed in dry climates where the groundwater level is low, and the chance of liquefaction is therefore low enough not to be considered a concern.

#### 2.3.5.2 Key metric for consideration

This topic does not apply.

#### 2.3.5.3 Infrastructure improvements

This topic does not apply.

#### 2.3.5.4 Cost-benefit considerations

This topic does not apply.

### *2.3.6 Wind hazard*

#### 2.3.6.1 General

Strong winds can damage PV modules. Proper design of the support frame, anchorage, and PV to support frame connections is required.

#### 2.3.6.2 Key metric for consideration

In typical design practice, static and dynamic uniform loading are used to design the system. However, the inclination of the PV panels can result in non-uniform loading, which has been shown to cause damage to the system.

#### 2.3.6.3 Infrastructure improvements

For solar farms, wind performance can be improved by several methods. Examples include:

- Design for the uplift that wind loading can cause; see Figure 20.
- Use a higher threshold than the code-minimum wind design to avoid the loss of solar panels (Eidinger et al. 2008).
- Install the panels horizontally to reduce wind demand.
- Design the support structure, anchorage, and connections for higher wind loads the code-minimum wind design.

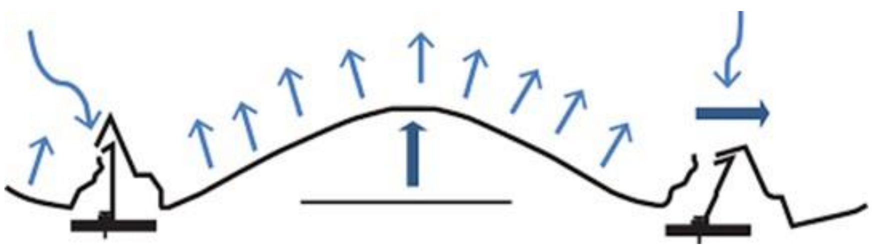


Figure 20. Design forces for wind uplift

#### 2.3.6.4 Cost-benefit considerations

For solar farms that are designed per code, with no factor of safety, there is a 20% chance of the need for replacement of solar farm equipment when the design wind speed occurs. This probability of failure is reduced by a factor of nearly 2.5, to 8%, when a more robust design (at approximately 15% higher cost) is implemented.

### *2.3.7 Flood hazard*

#### 2.3.7.1 General

Elevated solar farms are typically located away from wet areas and therefore the susceptibility is very low. However, in the case of wet area, damage states are defined as listed Table 7 (FEMA 2013c).

| **Overall vulnerability** | **Inundation** | **Scour** | **Hydraulic pressure & debris** | **Financial loss** | **Loss of operation** |
|---|---|---|---|---|---|
| Low | -- | Moderate | -- | -- | -- |

Table 7. Flood damage modes for elevated solar farms

### 2.3.7.2 Key metric for consideration

Local scour of the support frame columns could occur in the event of intense flash flooding. See Figure 21.

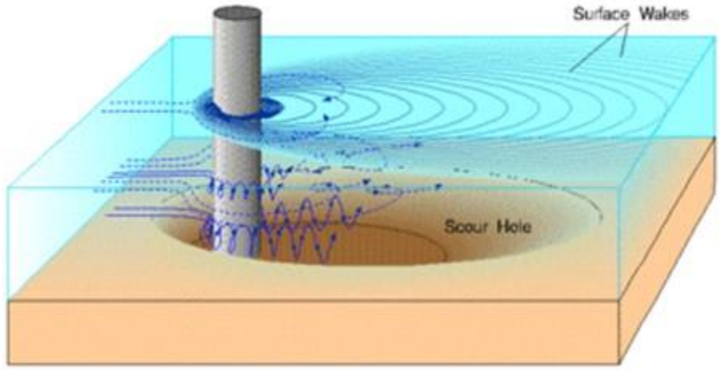



Figure 21. Local scour of vertical posts

### 2.3.7.3 Infrastructure improvements

Scour countermeasures include:

- Perform analysis and compute the potential scour depth.
- Ensure that the support columns have adequate strength and embedment to mitigate loss of soil due to scour.
- For existing units, increase the size of the footings.

### 2.3.7.4 Cost-benefit considerations

This topic does not apply.

## 2.4 Wind farms

### *2.4.1 Overview*

A wind farm, or “wind park,” is a group of wind turbines in the same location that are used to produce electricity. A wind turbine, alternatively referred to as a “wind energy converter,” is a device that converts the wind’s kinetic energy into electrical energy. Although turbines can be placed on land or offshore, this report mainly focuses on land-based units. Figure 22 shows an example of a wind farm.

Figure 22. Wind farm

### *2.4.2 Summary*

The results from a literature review of wind farms are summarized in Table 8 and are presented in the “*Improvement of infrastructure resilience evaluation matrix for selected natural hazards”* table of the project final report. The following subsections of this background report provide more detailed descriptions and background information.

| **Hazard** | **Susceptible** | **Improvements** | **Damage index** | | **Approximate improvement cost as % of cost** |
|---|---|---|---|---|---|
| | | | **As-is** | **Improved** | |
| **Earthquake** | Y | Better seismically designed units | 0.1 | 0.08 | 5% |
| **Liquefaction** | Y | Deep foundations | 0.2 | -- | 30% |
| **Wind** | Y | Better material for blade and gears | 0.2 | 0.1 | 5% |
| **Flood** | Very low | -- | -- | -- | -- |

Table 8. Summary of findings for wind farms

### *2.4.3 Vulnerability to natural hazards*

Wind farms have been moderately damaged in past natural hazard events. Figure 23 presents examples.

Earthquake (Japan)

Liquefaction (New Zealand)

Wind

This hazard does not apply.

Flood

Figure 23. Damage to wind farms from natural hazards

### *2.4.4 Earthquake hazard*

#### 2.4.4.1 General

Wind turbines are vulnerable to earthquakes and to the vertical component of ground motion due to their high frequency and relatively low damping.

#### 2.4.4.2 Key metric for consideration

Functionality of the system after an earthquake is a key metric for the impact on wind farms.

#### 2.4.4.3 Infrastructure improvements

For monopole wind turbines, earthquake performance can be improved by several methods. Examples include:

- Use improved analysis, including foundation radiation damping (soil-structure interaction).
- Check the performance for both operational and parked conditions.
- Consider the effect of soil and near-field motions for design, when applicable.
- Select a more seismically robust design.

#### 2.4.4.4 Cost-benefit considerations

Myers et al. (2012) examined the behavior of two 80-m-tall wind turbines with different capacity. Figure 24 presents the fragility functions for both units. Note that for the unit with better seismic performance, the 50% threshold for failure is reached at an earthquake intensity that is approximately 20% higher than the same threshold for the second unit.

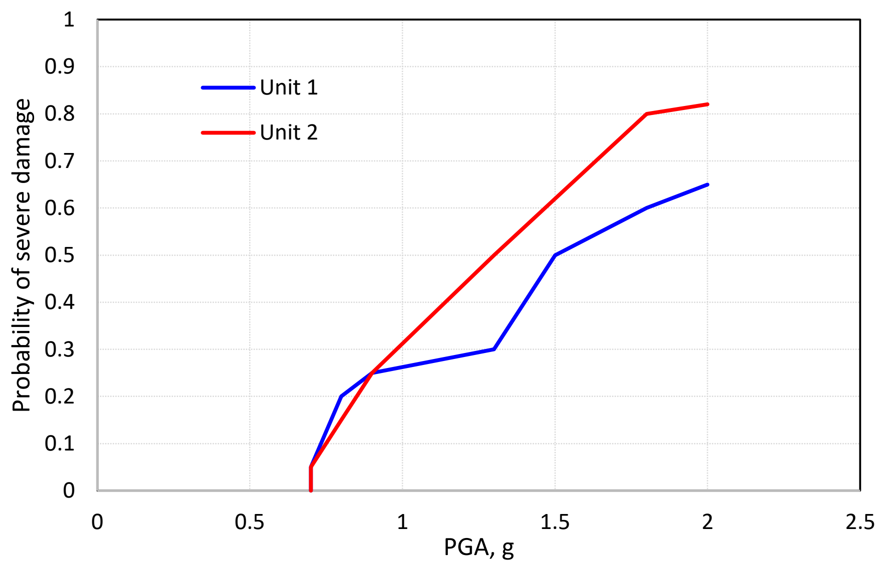



Figure 24. Damage fragility functions for two 80-m-tall wind turbines

Seismic improvements for wind turbines are cost-effective (estimated at 5% higher cost), given the level of expected improvement. In typical design, the probability of failure is approximately 10% for a large earthquake; with an improved design, this probability can be reduced to closer to 8%.

### *2.4.5 Liquefaction hazard*

#### 2.4.5.1 General

Wind turbines that are constructed on liquefiable soil can sustain damage.

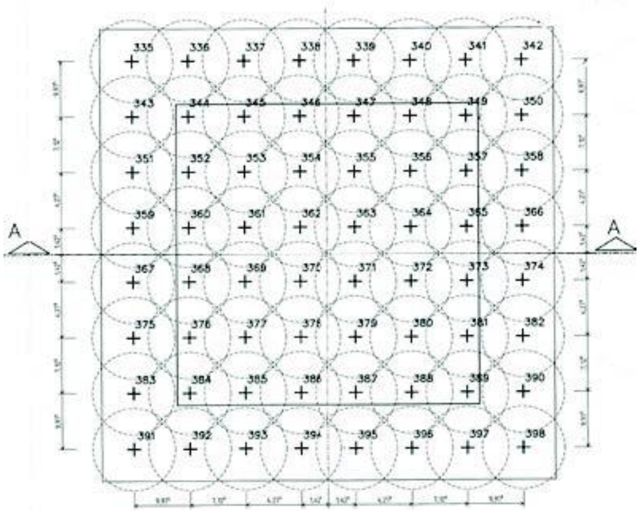
#### 2.4.5.2 Key metric for consideration

A key impact metric is the amount of time that it takes to repair wind turbines and to restore operations.

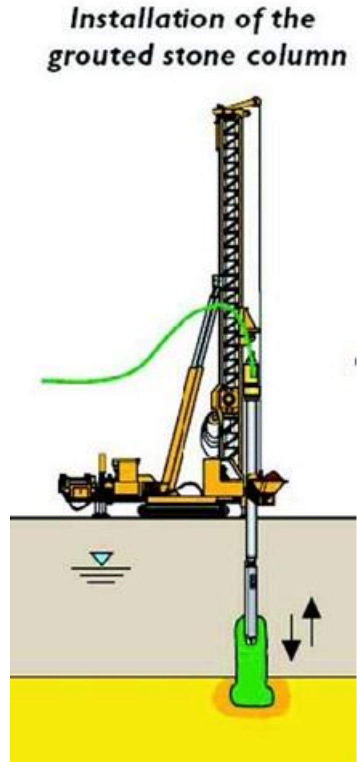
#### 2.4.5.3 Infrastructure improvements

For wind farms, liquefaction performance can be improved by the following:

- Use deep foundations; see Figure 25.



Vibro compaction




Vibro concrete column


Geopier

Post-tensioned concrete corrugated piles

Figure 25. Example of geotechnical liquefaction mitigation

### 2.4.5.4 Cost-benefit considerations

Wind turbines that are placed on liquefiable soil could tilt and cease to operate. Ground improvement or deep foundations are expensive. Wind farm owners must compare the efficacy of such retrofits with the power that is generated by such units. The added retrofit cost is estimated to be about 30% of the capital costs, and performance improvements include elimination of power loss (assuming a 20% probability of liquefaction failure without any improvements).

### *2.4.6 Wind hazard*

#### 2.4.6.1 General

Wind turbines rely on wind to generate power. However, they can also be susceptible to damage from wind loading that is caused by static and aerodynamic wind effects. The low inherent damping of the structure also exacerbates the wind response. In addition to large windstorms, failure could be caused by high-cycle fatigue due to wind and operational loading. Wind turbines are considered fatigue-critical structures. Failure of a blade can result in unbalanced forces acting on the pole and could result in overstress of the pole or local buckling of the pole upon impact from a failed blade.

Elements that contribute to fatigue consist of:

- Gravity force that causes tension and compression on blades during rotation
- Centrifugal forces from rotation
- Wind that is perpendicular to the blades
- Wind thrust

#### 2.4.6.2 Key metric for consideration

Structural reliability is used to assess the vulnerability of wind turbines. The number of wind cycles to failure is a parameter that determines the useful life of the turbine.

#### 2.4.6.3 Infrastructure improvements

For wind turbines, the fatigue life and thus the wind performance can be improved by several methods. Examples include:

- Use a blade design configuration that minimizes local stresses.
- Select a material with a high threshold for infinite fatigue life.
- Use a higher threshold than the code-minimum wind design to avoid fatigue failure.
- Design all connections and steel gear for high fatigue life.

#### 2.4.6.4 Cost-benefit considerations

Wind turbines use laminated wood, steel, aluminum, or composite (fiberglass). Steel has the best fatigue property; however, its use is limited because of its weight. The trend is toward using composite blades. Optimally, blades should be able to withstand $10^8$ cycles of loading before failure, providing a design life of 50 to 70 years. See Figure 26. Sutherland (1999) presents the S-N (number of cycles versus stress) curves for two composites. Note that the super composite (sample 1) can reach this threshold at a stress ratio of 0.2. However, for the second sample, the number of cycles is nearly 2.5 order of magnitude lower. The use of better material will add an estimated cost of 5% to the turbine design but can extend its life by a factor of 2 or more. After $10^6$ cycles, the superior material can carry nearly twice the stress of the second fiberglass, or its damage rate is approximately halved. In this report, the damage index for the superior material is assumed to be 10% for comparison.

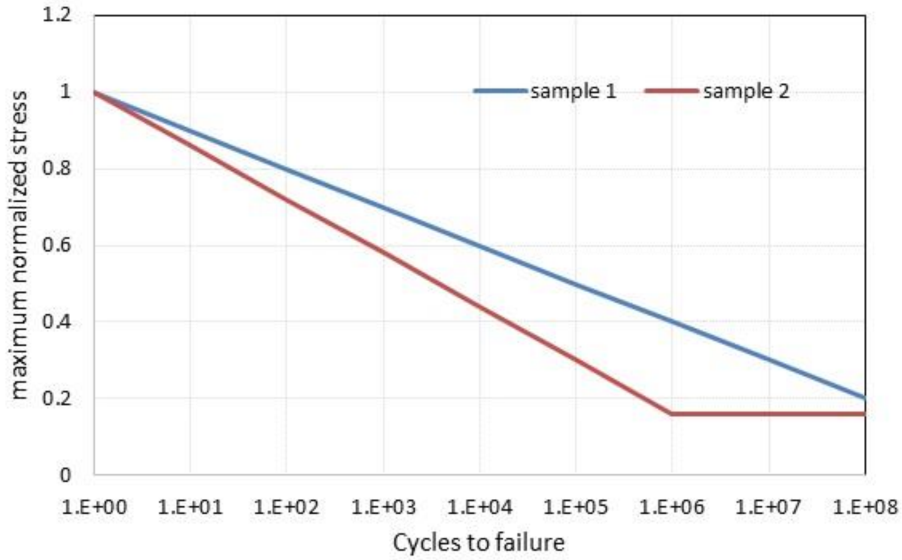



Figure 26. S-N curve for composite blade

### *2.4.7 Flood hazard*

#### 2.4.7.1 General

Tall wind turbines are typically not susceptible to flooding.

#### 2.4.7.2 Key metric for consideration

This topic does not apply.

#### 2.4.7.3 Infrastructure improvements

This topic does not apply.

#### 2.4.7.4 Cost-benefit considerations

This topic does not apply.

## 2.5 Nuclear power plants

### *2.5.1 Overview*

A nuclear power plant is a power station in which the heat from nuclear fission is used to operate turbines and generate electricity. Figure 27 shows an example of a nuclear power plant.

Figure 27. Nuclear power plant

### *2.5.2 Summary*

The results from a literature review of nuclear power plants are summarized in Table 9 and are presented in the "*Improvement of infrastructure resilience evaluation matrix for selected natural hazards"* table of the project final report. The following sections of this background report provide more detailed descriptions and background information.

| Hazard | Susceptible | Improvements | Damage index | | Approximate improvement cost as % of cost |
|---|---|---|---|---|---|
| | | | As-is | Improved | |
| **Earthquake** | Y | Better seismically designed units | 0.3 | 0.02 | 5% |
| **Liquefaction** | -- | -- | -- | -- | -- |
| **Wind** | -- | -- | -- | -- | -- |
| **Flood** | Low | Improved dike construction | 0.1 | 0.07 | 5% |

Table 9. Summary of findings for nuclear power plants

### *2.5.3 Vulnerability to natural hazards*

Nuclear power plants have been moderately damaged in past natural hazard events. Figure 28 presents examples.

Earthquake (Japan 2011)[†]

This hazard does not apply.

Liquefaction

This hazard does not apply.

Wind

Flood (Fort Calhoun NPP, United States, 2011)

Figure 28. Damage to nuclear power plants from natural hazards

### *2.5.4 Earthquake hazard*

#### 2.5.4.1 General

Nuclear power plants are vulnerable to earthquake damage. Fragility analysis is used for a structure, a system, or a component (SCC) to determine the probability of exceeding a limit state based on the capacity of the component. Typically, a 95% confidence level is used, which corresponds to the case in which 95% of median values are higher (Basu et al., 2015) and provides a safety margin.

#### 2.5.4.2 Key metric for consideration

Functionality and safety of the nuclear power plant and SCC of the system after an earthquake are key metrics to consider.

#### 2.5.4.3 Infrastructure improvements

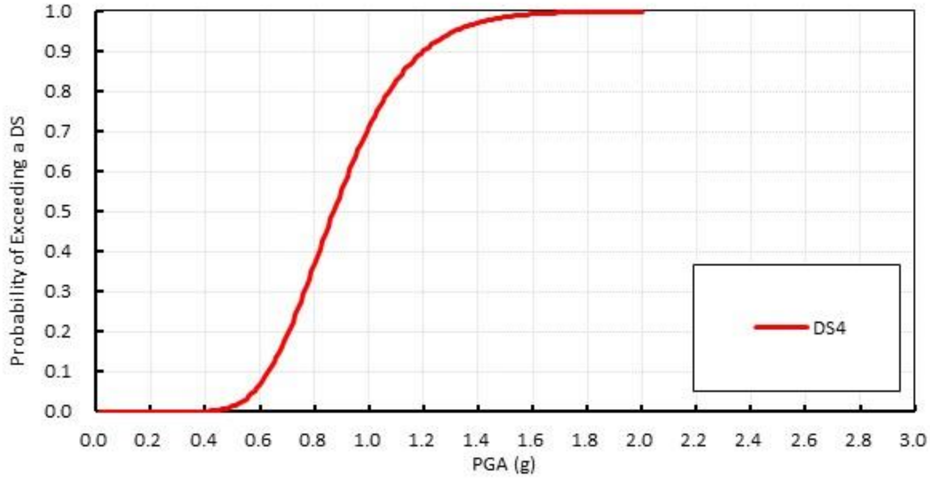
For nuclear power plants, earthquake performance can be improved by several methods. Examples include:

- Use seismic isolation.
- Ensure that all piping and conduit racks use flexible connections.
- Use a larger margin of safety.

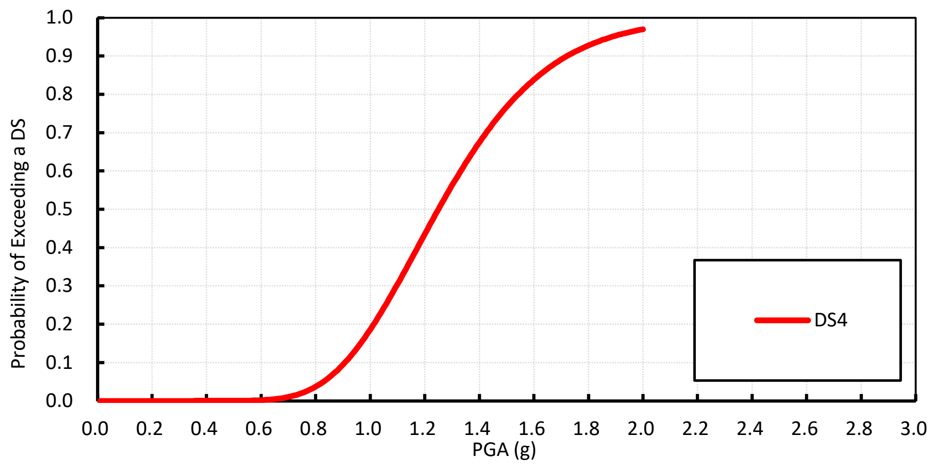
[†] Damage was from the tsunami that was caused by the earthquake and not from ground shaking.

### 2.5.4.4 Cost-benefit considerations

Figure 29 shows a fragility curve for an example fixed-based nuclear power plant (EPRI 2009) and for a seismically isolated nuclear power plant (Ahmed et al. 2015). Note that for an event with a PGA of 0.8g (a higher PGA is used here because nuclear power plants use a higher seismic design threshold than other power-generation facilities do), the probability of failure is reduced from approximately 30% for the fixed-base nuclear power plant to 2% for the seismically isolated plant. For the isolated case, the failure mode does not indicate system damage; instead, the isolator is reaching its displacement limit.




Fixed base




Isolated

Figure 29. Damage fragility functions for NPPs

Yu et al. (2018) compared cost evaluations for the seismic isolation of nuclear power plants. They found isolation to be cost-effective and that it can even lead to cost savings. In this report, the increase in cost is assumed 5% of capital costs, with the damage reduced by a factor of more than 10.

## *2.5.5 Liquefaction hazard*

### 2.5.5.1 General

Because of their strict design and concerns for safety, nuclear power plants are not constructed in liquefiable areas.

#### 2.5.5.2 Key metric for consideration

This topic does not apply.

#### 2.5.5.3 Infrastructure improvements

This topic does not apply.

#### 2.5.5.4 Cost-benefit considerations

This topic does not apply.

### *2.5.6 Wind hazard*

#### 2.5.6.1 General

Nuclear power plants are not susceptible to wind loading. Other components and buildings on site can be damaged from wind forces and must be designed as discussed earlier in this report, for example see Section 2.1.6.1. Nuclear power plants are not vulnerable to winds because:

- The outer wall of a reactor is designed to withstand a large commercial airliner and thus can withstand flying debris in hurricanes.
- Sometimes plants are shut down before the arrival of windstorms or hurricanes.

#### 2.5.6.2 Key metric for consideration

This topic does not apply.

#### 2.5.6.3 Infrastructure improvements

This topic does not apply.

#### 2.5.6.4 Cost-benefit considerations

This topic does not apply.

### *2.5.7 Flood hazard*

#### 2.5.7.1 General

Some NPPs are vulnerable to flooding that is caused by external events, such as the units that are near coastal areas. Typically, dikes are constructed to protect against flooding.

#### 2.5.7.2 Key metric for consideration

Water overtopping or seeping through the dike and flooding the NPP, causing damage and loss of operation, is a key metric to consider. Such damage is measured by using external flood fragility functions (Bensi et al. 2015).

#### 2.5.7.3 Infrastructure improvements

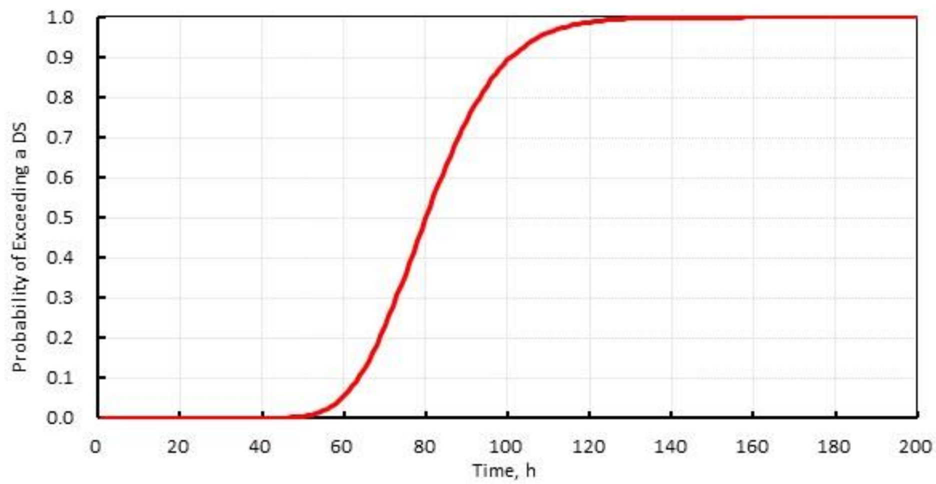
For NPPs, external flood performance can be improved by several methods. Examples include:

- Use taller dikes.
- Ensure that the design is based on an extreme flood event and not on a design-level flood.
- Account for a tsunami in the aftermath of earthquakes.
- Ensure that the dike is constructed on competent soil.

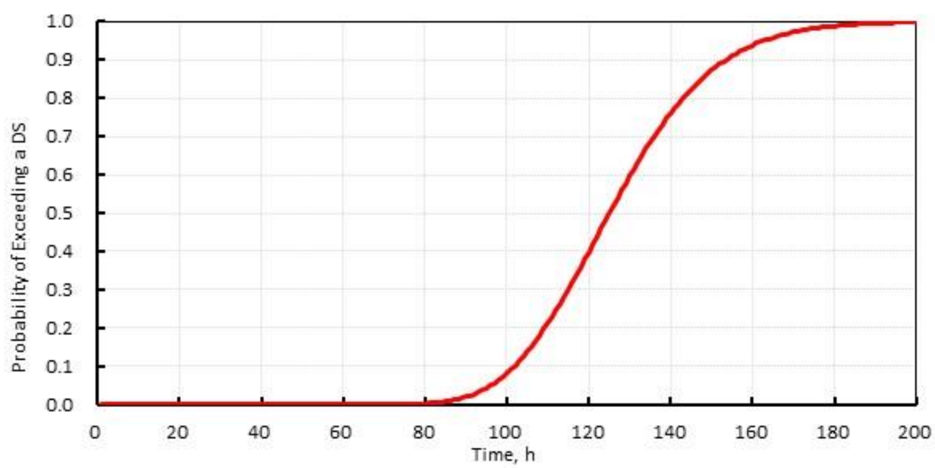
#### 2.5.7.4 Cost-benefit considerations

Vorogushyn et al. (2009) performed flood fragility analysis of dikes, looking at several parameters and failure modes. Figure 30 presents the fragility functions for cases of piping that forms at rates of 1.5 m and

1 m per day, respectively. Note that for more competent soil (Situation 2) in which the pipe development rate is reduced, the median time to form the pipe increased from approximately 80 hours to 120 hours. Assuming that damage is proportional to anomalies in the dike, the damage rate is reduced by a factor of 30% with more competent soil. For nominal design, a damage probability of 10% is assumed. By improving dike construction with an estimated 5% additional cost, the damage probability is then reduced to 7%.




Situation 1




Situation 2

Figure 30. Flood fragility functions for two dikes

## 2.6 Substations

### *2.6.1 Overview*

An electrical substation is a facility that provides a source of electricity for the community that it serve. Figure 31 shows an example of a substation. Electrical substations convert power from one voltage to another and serve as a key component that links power-generation stations to customers.

Figure 31. High-voltage electrical substation

### *2.6.2 Summary*

The results from a literature review of electrical substations are summarized in Table 10 and are presented in the "*Improvement of infrastructure resilience evaluation matrix for selected natural hazards"* table of the project final report. The following subsections of this background report provide more detailed descriptions and background information.

| Hazard | Susceptible | Improvements | Damage index | | Approximate improvement cost as % of cost |
|---|---|---|---|---|---|
| | | | As-is | Improved | |
| **Earthquake** | Y | Anchorage, seismic components | 0.8 | 0.3 | 10% |
| **Liquefaction** | Y | Deep foundation | 0.6 | Low | 20% |
| **Wind** | Y | Higher factor of safety | 0.3 | 0.1 | 20% |
| **Flood** | Y | Elevated components | 0.1 | Low | 10% |

Table 10. Summary of findings for electrical substations

### *2.6.3 Vulnerability to natural hazards*

Electrical substations have been significantly damaged in past natural hazard events. Figure 32 presents examples.

Because the worldwide trend is toward using higher-voltage (500 to 1,100 kV) components, data for high-voltage assets is presented in this report.

Earthquake (Northridge Earthquake)

Liquefaction (Christchurch Earthquake)

Wind (Hurricane Harvey)

Flood (aftermath of Hurricane Matthew)

Figure 32. Damage to high-voltage substations from natural hazards

### *2.6.4 Earthquake hazard*

#### 2.6.4.1 General

Electrical substations are susceptible to damage from peak ground acceleration (PGA). Damage states are defined as listed in Table 11 (FEMA 2013a).

| Damage state | Definition | Status | Power | Restoration, days (median) |
|---|---|---|---|---|
| DS0 (none) | -- | Operational | Normal | 0 |
| DS1 (minor) | Failure of 5% of the disconnect switches (e.g., misalignment), or failure of 5% of the circuit breakers (e.g., circuit breaker phase sliding off its pad, circuit breaker tipping over, or interrupter-head falling to the ground), or the building being in minor damage state | Operational | Close to nominal | 1 |
| DS2 (moderate) | Failure of 40% of disconnect switches (e.g., misalignment), or 40% of circuit breakers (e.g., circuit breaker phase sliding off its pad, circuit breaker tipping over, or interrupter-head falling to the ground), or | Operational without repair | Reduced | 3 |

| Damage state | Definition | Status | Power | Restoration, days (median) |
|---|---|---|---|---|
| | failure of 40% of current transformers (e.g., oil leaking from transformers, porcelain cracked), or the building being in moderate damage state | | | |
| DS3 (extensive) | Failure of 70% of disconnect switches (e.g., misalignment), 70% of circuit breakers, 70% of current transformers (e.g., oil leaking from transformers, porcelain cracked), or failure of 70% of transformers (e.g., leakage of transformer radiators), or the building being in extensive damage state | Operational after repair | No | 7 |
| DS4 (complete) | Failure of all disconnect switches, all circuit breakers, all transformers, or all current transformers, or the building being in complete damage state | Not repairable | No | 30 |

Table 11. Earthquake damage states for electrical substations

### 2.6.4.2 Key metric for consideration

A key impact metric for electrical substations is the amount of time that it takes to restore operations. Figure 33 shows the timeline of restoration (FEMA 2013a) for electrical substations.

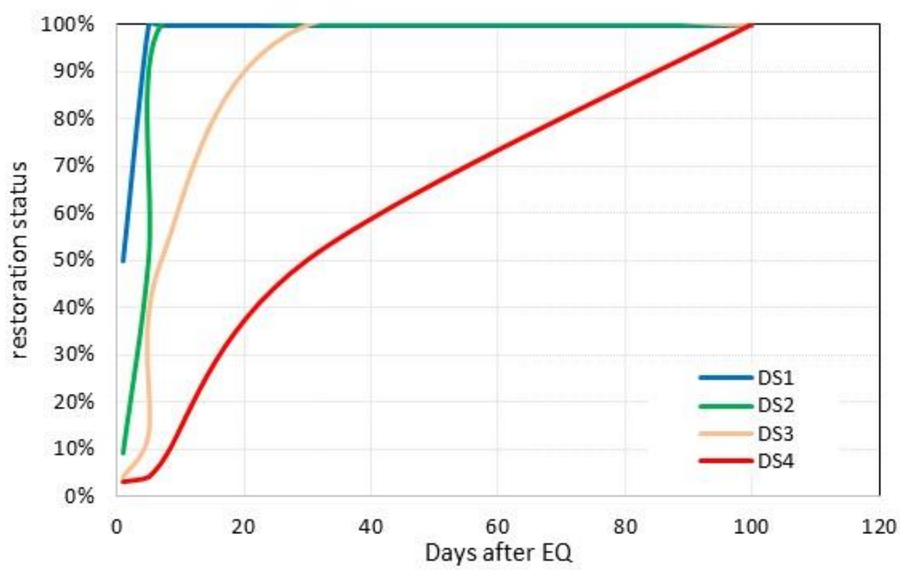



Figure 33. Restoration curve, electrical substations, earthquake hazard

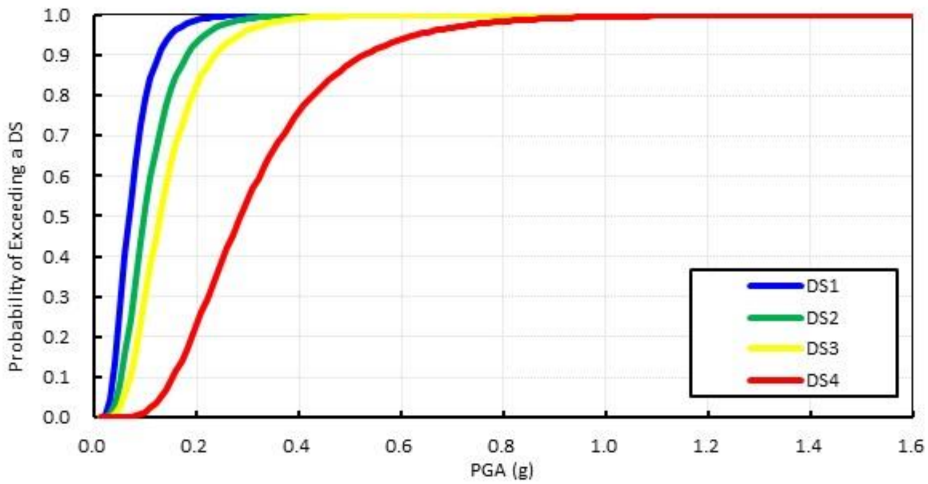
### 2.6.4.3 Infrastructure improvements

For electrical substations, earthquake performance can be improved by several methods. Examples include:

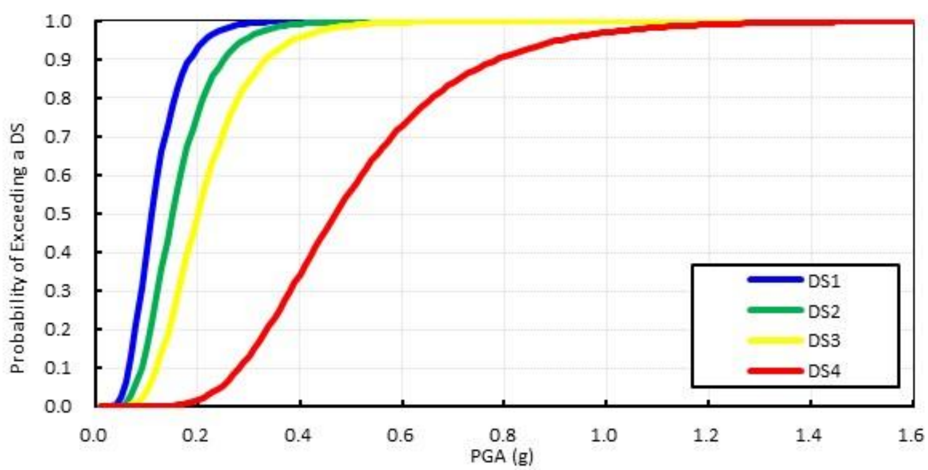
- Provide proper anchorage of all components, including transformers.
- Use only tested or analyzed components that have shown robustness during evaluation.
- Use seismic protection devices to reduce demand on the components.
- Ensure that backup units are available that can be installed quickly if a component such as a building or a circuit breaker is damaged.

### 2.6.4.4 Cost-benefit considerations

FEMA (2013a) provides fragility functions for substations in use in the United States. Two sets of data are provided: one for substations that have standard components and one for substations that have proper anchorage and seismic components. In this report, the fragility parameters are modified for the unanchored case to account for the higher variability and the lower expected quality in worldwide application. Figure 34 presents the fragility functions for non-seismic and enhanced electrical substations, respectively. Note that for an earthquake with a PGA of 0.4g, the probability of exceeding DS4 is 80% for the non-seismic substation and is 30% for the seismic substation. In other words, by using seismic components (albeit at a slightly higher cost, estimated at approximately 10% in this report), which will improve seismic performance and likely lower the damage status, the time-to-restoration period (e.g., 30 days) would be much shorter than for the non-seismic case. Such expedited restoration provides significant economic benefits and helps in the community recovery.




Unanchored subcomponents




Anchored and seismic subcomponents

Figure 34. Damage fragility functions for high-voltage electrical substations

### *2.6.5 Liquefaction hazard*

#### 2.6.5.1 General

Electrical substations that are constructed on sandy soils in earthquake zones are vulnerable to damage from liquefaction.

#### 2.6.5.2 Key metric for consideration

Structural damage is used to assess the vulnerability of electrical substation components. Liquefaction can damage electrical substations and result in loss of operation. Because the failure is below ground, the cost of geotechnical repair and restoration of operations is high.

#### 2.6.5.3 Infrastructure improvements

For electrical substations, liquefaction performance can be improved by several methods. Examples include:

- Use deep foundations; see Figure 35.
- Perform soil grouting and densification.

Figure 35. Example of liquefaction mitigation

#### 2.6.5.4 Cost-benefit considerations

For electrical substations that are constructed on a shallow foundation, liquefaction can result in damage and loss of operation and power for an extended period. Power restoration would be costly and long delayed. By contrast, electrical substations that are constructed by using deep foundations (piles) driven past the liquefiable layers would be immune to such extensive damage. Assuming that the substructure cost is 20% of the total substation cost, a deep foundation could add approximately 20% to the construction cost. This foundation cost is significantly lower than the long-term loss of revenue that liquefaction damage can cause to substations.

### *2.6.6 Wind hazard*

#### 2.6.6.1 General

The tall components (such as disconnect switches) of electrical substations are susceptible to damage from wind. Such damage can result in loss of operation of the substation.

### 2.6.6.2 Key metric for consideration

Structural reliability is used to assess the vulnerability of electrical substation components. Structural reliability is the probability that the structure will not reach a limit state (e.g., a failure state) during a given period of strong winds. With this approach, the responses of various substation components are averaged out, thus it eliminates some of the uncertainties.

Figure 36 (Lopez et al. 2009) presents the probability of failure as a function of wind speed for a 400-kV substation that is designed for 200-km/hr. winds. Note that for this substation, there is an approximately 30% chance of failure when a windstorm with the threshold wind intensity occurs.

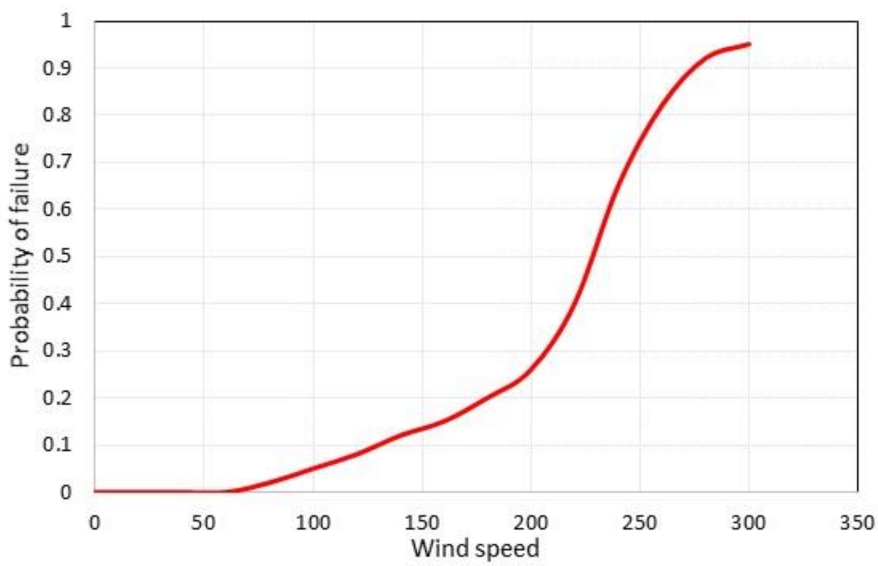



Figure 36. Probability of failure as a function of wind speed (km/hr.)

### 2.6.6.3 Infrastructure improvements

For electrical substations, wind performance can be improved by several methods. Examples include:

- Design the substation components for a higher design wind speed; see Figure 37 (Lopez et al. 2009).
- Ensure that all components and buildings are properly anchored by using positive mechanical attachments that are wind-rated.
- Positively anchor all substation buildings roofs to columns and walls.

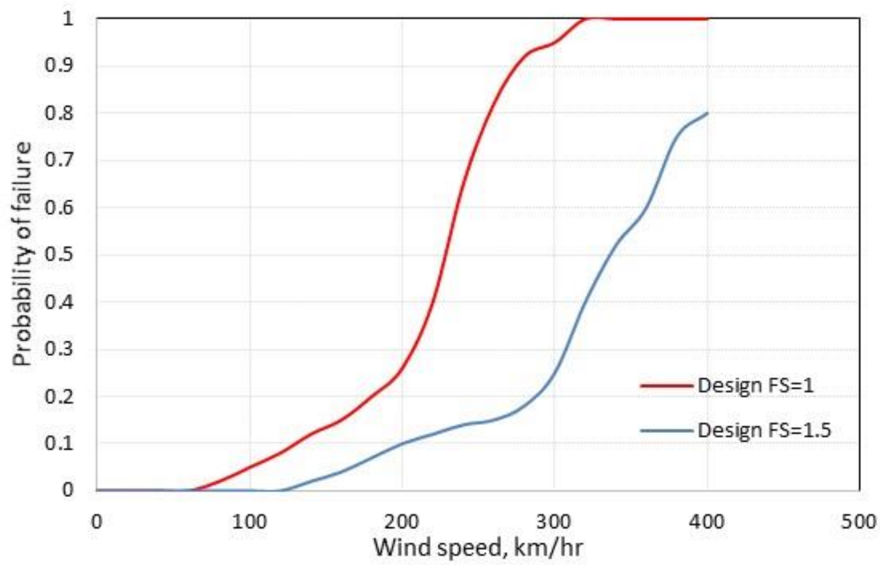



Figure 37. Example of wind mitigation (km/hr.) for different factors of safety

### 2.6.6.4 Cost-benefit considerations

For electrical substations that are designed per code, with no factor of safety, there is a 30% chance of substation failure when a windstorm with the threshold wind speed occurs. This probability of failure is reduced by a factor of nearly 3 when a more robust design (at a higher cost) is implemented.

## *2.6.7 Flood hazard*

### 2.6.7.1 General

Electrical substations are susceptible to damage from flooding. Flooding can damage expensive components or other components that, when damaged, lead to extended loss of power. Electrical substations are particularly vulnerable to flooding. Vulnerabilities are defined as listed in Table 12 (FEMA 2013c).

| **Overall vulnerability** | **Inundation** | **Scour** | **Hydraulic pressure & debris** | **Financial loss** | **Loss of operation** |
|---|---|---|---|---|---|
| High | High | -- | -- | High | High |

Table 12. Flood vulnerability for electrical substations

### 2.6.7.2 Key metric for consideration

The status of damage (FEMA 2013c) for electrical substations is presented in Figure 38. At the onset of flooding, damage to the control room starts and is maximized at a water level of 7 ft. Additional damage to cabling and incidental damage to transformers and switchgear occur. In many instances, the stations will be shut down based on a predetermined floodwater-level threshold.

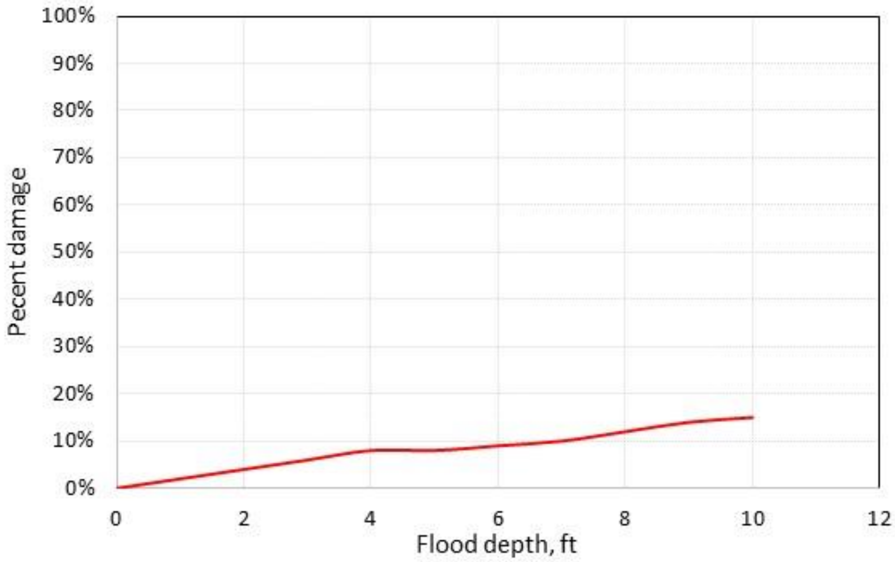



Figure 38. Percentage of damage as a function of inundation depth

### 2.6.7.3 Infrastructure improvements

For electrical substations, flood performance can be improved by several methods. Examples include:

- Install flood-monitoring devices to notify operators when flooding occurs and as it reaches certain levels.
- Elevate components.
- Provide flood-protection walls or gates; see Figure 39.


Elevation of components

Floodgates

Figure 39. Examples of flood mitigation

### 2.6.7.4 Cost-benefit considerations

FEMA (2013c) provides flood damage data for substations in use in the United States. For substations without protection, flood inundation of 7 ft. implies a maximum state for the control room and a damage state of 10%. For typical substations, this damage level could be approximately 10% of the substation cost

(US $5 million). In 2013, 32 power stations in New York were raised above the flood line, for a cost of US $72 million, or approximately US $2.3 million per substation or 5% of the infrastructure cost. Alternatively, concrete walls or floodgates can be used at similar level of expenditure and provide cost-effective solutions.

## 2.7 Transmission and distribution (T&D) systems

### *2.7.1 Overview*

The purpose of a transmission and distribution (T&D) system is to transfer electrical energy from generating units at various locations to the customers who need the power. Figure 40 shows an example of a T&D system. T&D systems comprise high voltage distribution that spans long distances and a medium-to-low-voltage local distribution system. The distribution system can be both overhead and underground. Most high-voltage systems are overhead, and this section focuses on those systems. Underground T&D systems perform similarly to underground water distribution systems, which are discussed in Section 4.4 Distribution Pipes.

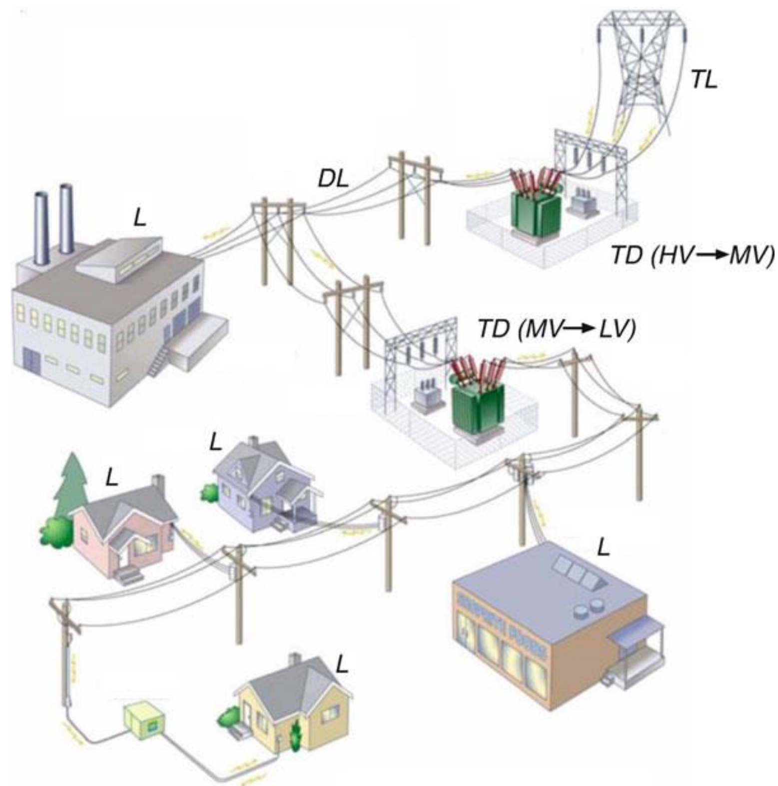



Figure 40. T&D system

### *2.7.2 Summary*

The results from a literature review of T&D systems are summarized in Table 13 and are presented in the "*Improvement of infrastructure resilience evaluation matrix for selected natural hazards"* table of the project final report. The following subsections of this background report present more detailed descriptions and background information.

| Hazard | Susceptible | Improvements | Damage index | | Approximate improvement cost as % of cost |
|---|---|---|---|---|---|
| | | | As-is | Improved | |
| **Earthquake** | Low | Seismic components | 0.02 | .01 | 2% |
| **Liquefaction** | Y | Deep foundation | 0.2 | 0.01 | 15% |
| **Wind** | Y | Steel, concrete, composite towers and poles | 0.3 | 0.07 | 20% |
| **Flood** | N | -- | -- | -- | -- |

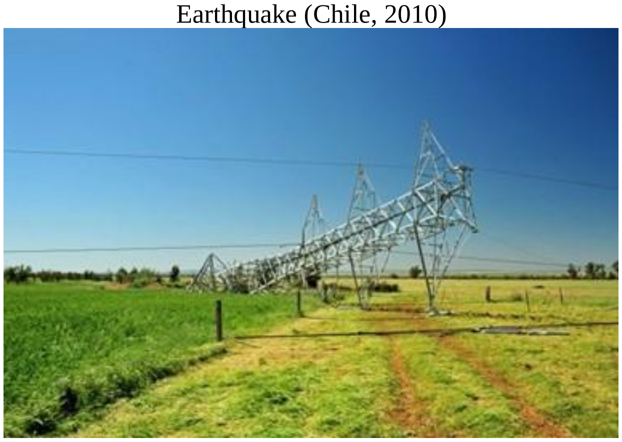
Table 13. Summary of findings for T&D systems

### 2.7.3 Vulnerability to natural hazards

T&D systems have been significantly damaged in past natural hazard events. Figure 41 presents examples.

Earthquake (Chile, 2010)

Liquefaction (Christchurch Earthquake, 2011)



Wind (South Australia, 2016)

This hazard does not apply.

Flood

Figure 41. Damage to high-voltage T&D systems from natural hazards

### 2.7.4 Earthquake hazard

#### 2.7.4.1 General

T&D systems are not particularly susceptible to damage from peak ground acceleration (PGA). Damage states are defined as listed in Table 14 (FEMA 2013a).

| Damage state | Definition |
|---|---|
| DS0 (none) | -- |
| DS1 (minor) | Failure defined by the failure of 4% of all circuits. |
| DS2 (moderate) | Failure is defined by the failure of 12% of circuits. |
| DS3 (extensive) | Failure is defined by the failure of 50% of all circuits. |

| Damage state | Definition |
|---|---|
| DS4 (complete) | Failure is defined by the failure of 80% of all circuits. |

Table 14. Earthquake damage states for overhead T&D systems

#### 2.7.4.2 Key metric for consideration

A key impact metric for T&D systems is the amount of time that it takes to restore operations. Figure 42 shows the timeline of restoration (FEMA 2013a) for electrical T&D systems.

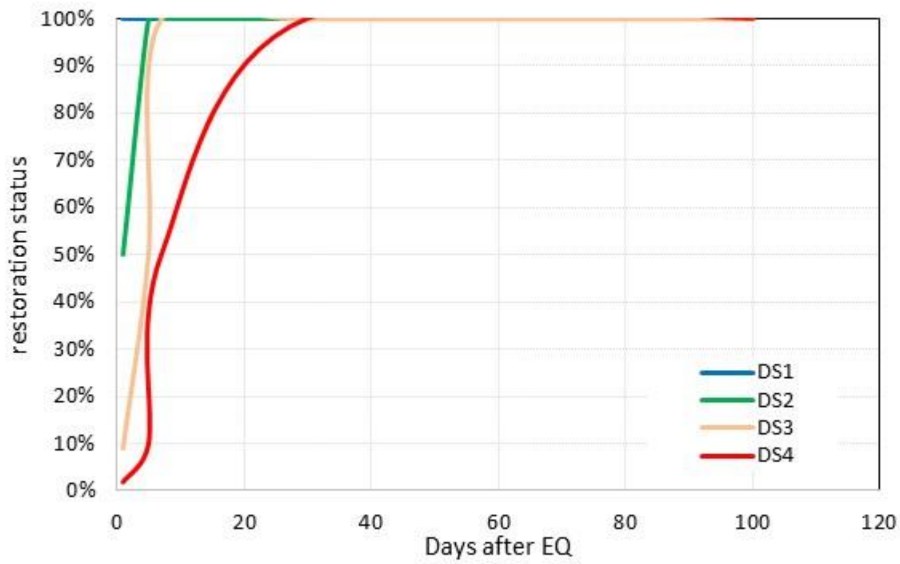



Figure 42. Restoration curve, T&D systems, earthquake hazard

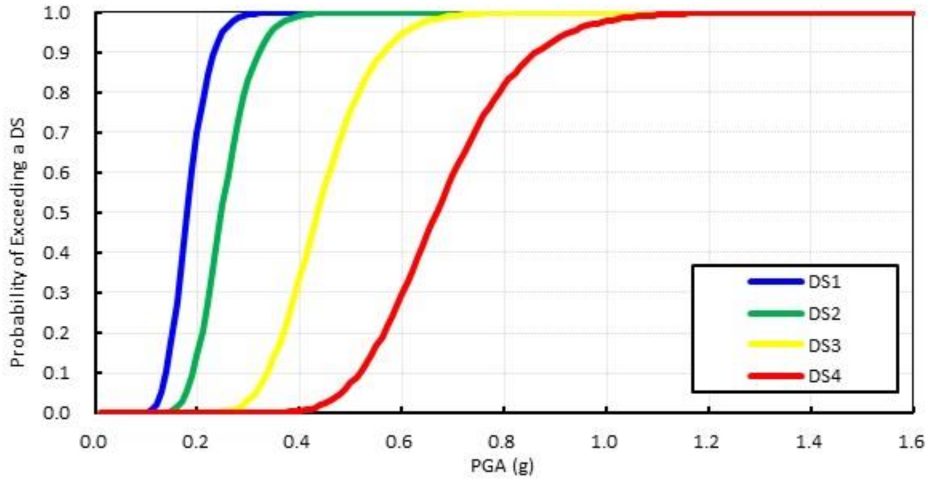
#### 2.7.4.3 Infrastructure improvements

For T&D systems, earthquake performance can be improved by the following:

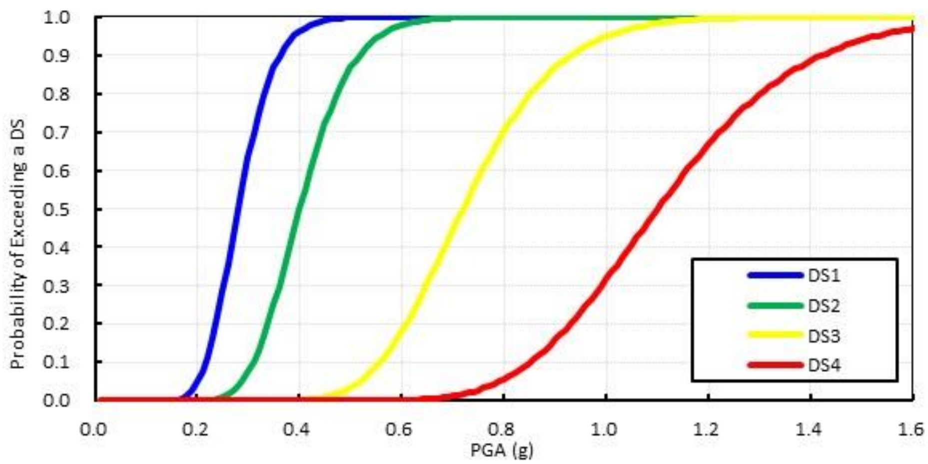
- Use seismic components.

#### 2.7.4.4 Cost-benefit considerations

FEMA (2013a) provides fragility functions for T&D systems in use in the United States. Two sets of data are provided: one for T&D systems that have standard components and one for T&D systems that have seismic components. In this report, the fragility parameters are modified for the unanchored case to account for the higher variability and lower expected quality in worldwide application. Figure 43 presents the fragility functions for standard and seismic components, respectively. Note that for an earthquake with a PGA of 0.4g, the probability of exceeding DS4 is 2% for the standard T&D system and is 1% for the seismic T&D system. In other words, by using seismic components (albeit at a slightly higher cost, estimated at approximately 2% in this report), which will improve seismic performance and likely lower the damage status, the time-to-restoration period (e.g., 7 days) would be much shorter than for the non-seismic case. During decision making, T&D system owners must compare the extra cost of upgrades with the benefits.




Standard design




Seismic design

Figure 43. Damage fragility functions for high-voltage electrical T&D systems

### *2.7.5 Liquefaction hazard*

#### 2.7.5.1 General

T&D lines that are anchored in liquefiable soil in earthquake zones are vulnerable to damage.

#### 2.7.5.2 Key metric for consideration

Structural damage is used to assess the vulnerability of T&D systems. Liquefaction can damage T&D systems and cause loss of continuity, which can have a cascading effect and lead to failure of adjacent towers.

#### 2.7.5.3 Infrastructure improvements

For T&D systems, liquefaction is mitigated by the following:

- Use deep foundations; see Figure 44 (Doohyun et al. 2015).

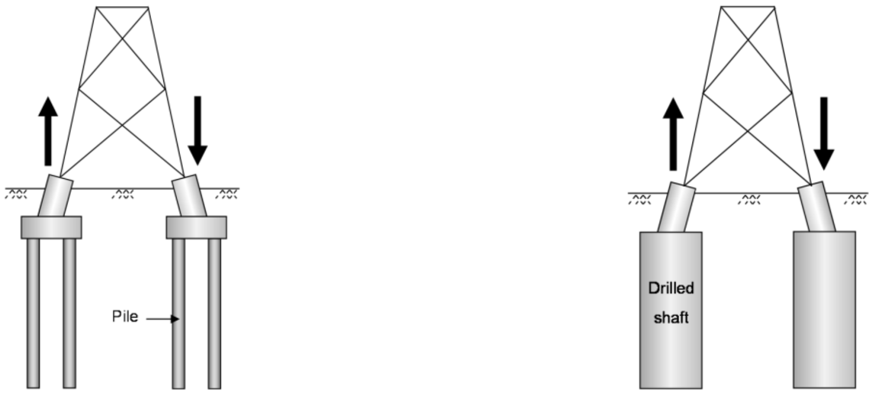



Figure 44. Examples of liquefaction mitigation for T&D systems

### 2.7.5.4 Cost-benefit considerations

For T&D towers that are constructed on a shallow foundation, liquefaction can result in damage and loss of operation and power for an extended period. Power restoration could be costly and long delayed. By contrast, towers that are constructed by using deep foundations (piles) driven past the liquefiable layers would be immune to such extensive damage. A deep foundation could add approximately 15% to the construction cost. This foundation cost is lower than the long-term loss of revenue that liquefaction damage can cause to T&D systems, however.

## *2.7.6 Wind hazard*

### 2.7.6.1 General

Tall and slender (and low-damped) T&D towers are susceptible to damage from wind. Such damage can result in loss of power.

### 2.7.6.2 Key metric for consideration

Structural reliability is used to assess the vulnerability of T&D system components. With this approach, the responses of various T&D system components are averaged out, thus it eliminates some of the uncertainties. Figure 45 (Lopez et al. 2009) presents the probability of failure as a function of wind speed for a tall tower that is designed for 120-km/hr. winds. Note that for this tower, failure is not expected at the threshold wind speed.

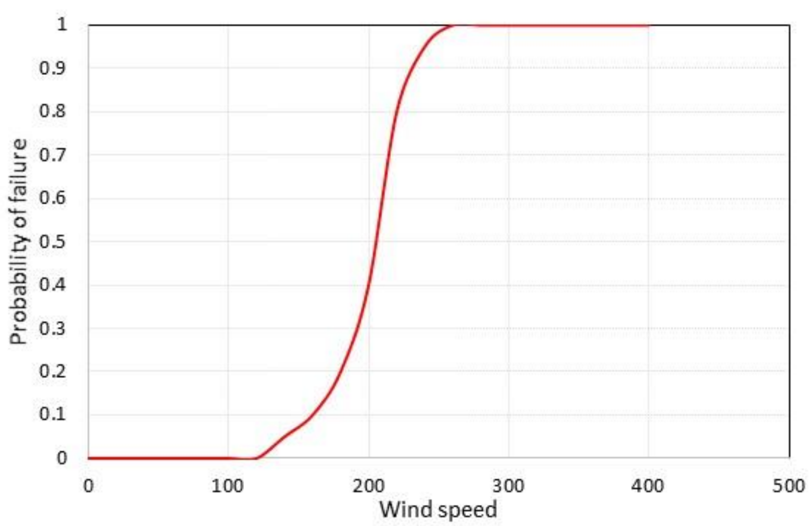



Figure 45. Probability of failure as a function of wind speed (km/hr.)

### 2.7.6.3 Infrastructure improvements

For T&D systems, wind performance can be improved by several methods. Examples include:

- Design the T&D system components for a higher design wind speed; see Figure 46 (Lopez et al. 2009).
- For distribution systems, this more resilient design usually involves upgrading wooden poles to concrete, to steel, or to a composite material and installing support wires and other structural supports.
- For transmission systems, this more resilient design usually involves upgrading aluminum structures to galvanized steel lattice or to concrete.
- Add more transmission lines to increase power flow capacity and to provide greater control over energy flows. By increasing the ability to bypass damaged lines and to reduce the risk of cascading failures, this approach can increase system flexibility.

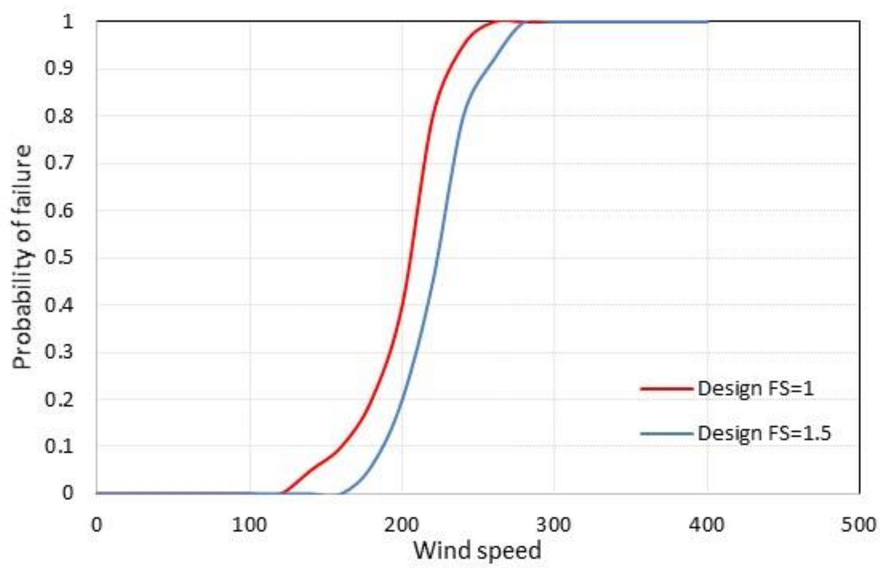



Figure 46. Example of wind mitigation (km/hr)

### 2.7.6.4 Cost-benefit considerations

For T&D systems that are designed per code, with no factor of safety, there is 30% chance of T&D system failure when a windstorm with 1.5 times the threshold wind speed occurs. This probability of failure is reduced by a factor of more than 4 when a more robust design (at a higher cost) is implemented.

## *2.7.7 Flood hazard*

### 2.7.7.1 General

Overhead T&D systems are only modestly vulnerable to flood damage. Vulnerabilities are defined as listed in Table 15 (FEMA 2013c).

| **Overall vulnerability** | **Inundation** | **Scour** | **Hydraulic pressure & debris** | **Financial loss** | **Loss of operation** |
|---|---|---|---|---|---|
| Low | -- | Medium | Low | Low | Low |

Table 15. Flood vulnerability for T&D systems

### 2.7.7.2 Key metric for consideration

Figure 47 presents the status of damage (FEMA 2013c) for overhead T&D systems. As the graph shows, little damage is anticipated.

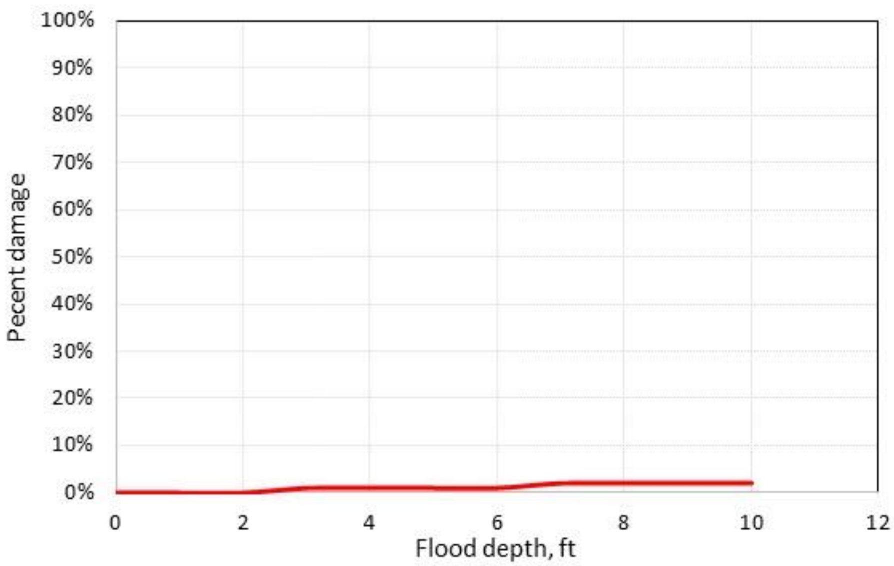



Figure 47. Percentage of damage as a function of inundation depth

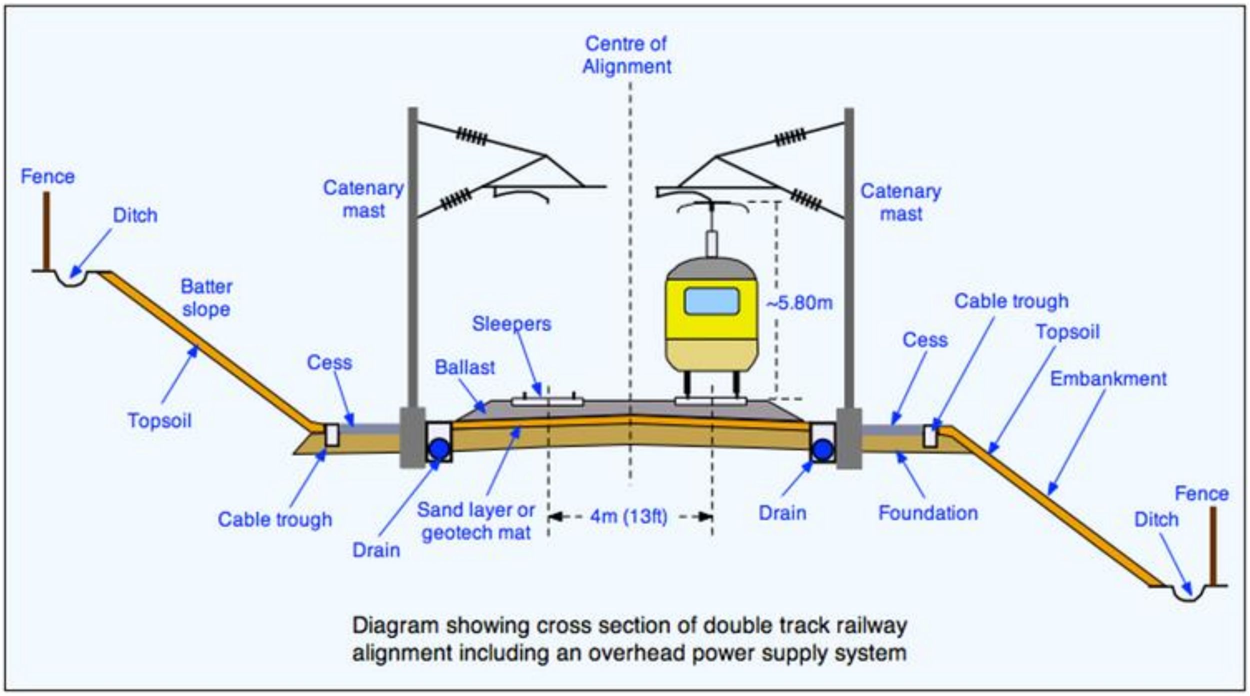
### 2.7.7.3 Infrastructure improvements

This topic does not apply.

### 2.7.7.4 Cost-benefit considerations

This topic does not apply.

## 3. TRANSPORT INFRASTRUCTURE



## 3.1 Railways (diesel and electric)

### *3.1.1 Overview*

A railway system is one of several types of transportation networks and is a necessary infrastructure component in a non-automobile society. A railway system typically is spread widely, so the dominant natural hazard at each location changes depending on each natural environment. In addition, because a railway system is composed of many kinds of components, the critical aspects of each natural hazard differ. A railway system, shown in Figure 48, can be expressed as a simplified network that consists of seven representative components. Those components include tracks/roadbeds, bridges, tunnels, railway stations (buildings), fuel/DC substations, dispatch facilities (buildings), and maintenance facilities (buildings) (FEMA 2013a & AREMA 2018).

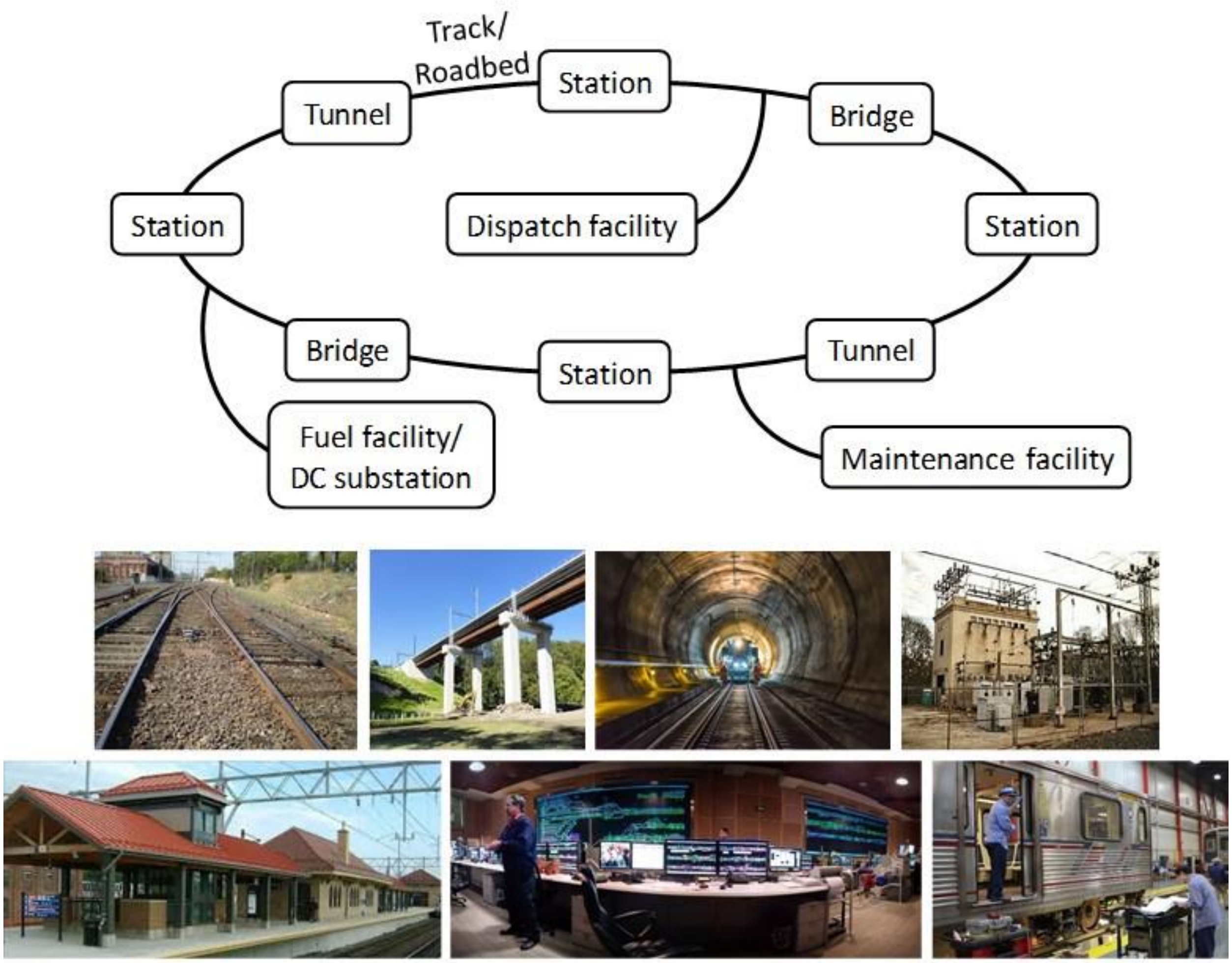



Figure 48. Railway systems and their components

### *3.1.2 Summary*

The results from a literature review for railways are summarized in Table 16 and are presented in the "*Improvement of infrastructure resilience evaluation matrix for selected natural hazards"* table of the project final report. The improvement cost in this table is simply shown as the ratio of the improvement cost to the component replacement cost, and the resiliency index is estimated as a probability of exceeding severe damage (i.e., more than severe) when the hazard threshold intensity occurs. The following subsections of this background report provide more detailed descriptions and background information.

| Hazard | Susceptible | Improvements | Resiliency index | | Approximate improvement cost as % of cost |
|---|---|---|---|---|---|
| | | | As-is | Improved | |
| **Earthquake** | Y | Bridge pier jacketing | 0.12 | 0.05 | 25% |
| **Liquefaction** | Y | Drainage & drainpipe installation | 0.16 | 0.01 | 45% |
| **Wind** | Y | Connection & envelope retrofit | 0.04 | 0.03 | 15% |
| **Flood** | Y | Elevation & barrier installation | 0.03 | 0.01 | 50% |

Table 16. Summary of findings for railways

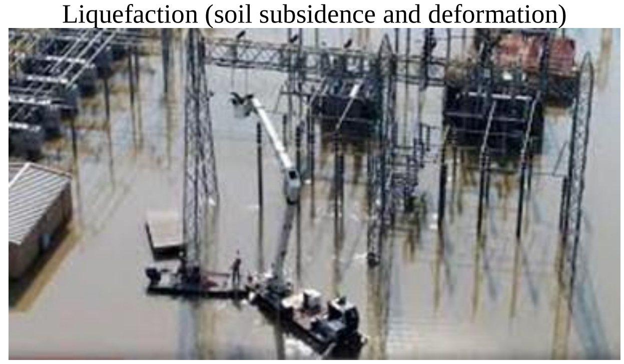
### *3.1.3 Vulnerability to natural hazards*

Railway systems have been significantly damaged in past natural hazard events, as exemplified in Figure 49. When a design-level earthquake occurs, major damage to railway systems typically occurs at bridge piers; therefore, these piers are one of the most critical components during seismic shaking (Day and Barkan 2002). When liquefaction occurs, the track/roadbed is usually vulnerable to ground displacement and failure, making it a critical component (Day and Barkan 2002 & AREMA 2018). In a wind hazard, a building is the most critical component because it has a large area that is loaded by wind, such as the roof and walls, especially for a building with more architectural openings, such as a railway station. In a flood disaster, mechanical/electrical equipment becomes the most critical component; because this kind of equipment is generally weak in water and is automatically shut down to avoid secondary dangers. Therefore, this type of equipment at a fuel/DC substation of the railway system is assumed the most critical component in a flood disaster (NYT 2012 & NYC 2013).

Earthquake (bridge pier collapse)

Liquefaction (soil subsidence and deformation)


Wind (roof and wall damage)



Flood (substation inundation)

Figure 49. Damage to railway systems from natural hazards

### *3.1.4 Earthquake hazard*

#### 3.1.4.1 General

The level of seismic damage to a bridge pier is largely affected by peak ground acceleration (PGA) due to earthquake shaking. The damage states of a bridge pier can be generally defined as listed in Table 17 (FEMA 2013a).

| Damage state | Definition | Status | Functionality |
|---|---|---|---|
| DS0 (none) | -- | Operational | Normal |
| DS1 (minor) | Minor cracking and spalling to the abutment, cracks in shear keys at abutments, minor spalling and cracks at hinges, minor spalling at the column or minor cracking to the deck | Operational | Close to normal |
| DS2 (moderate) | Any column experiencing moderate (shear cracks) cracking and spalling (column structurally still sound), moderate movement of the abutment (<2"), extensive cracking and spalling of shear keys, any connection having cracked shear keys or bent bolts, keeper bar failure without unseating, rocker bearing failure or moderate settlement of the approach | Operational with minor repair | Reduced |
| DS3 (extensive) | Any column degrading without collapse - shear failure - (column structurally unsafe), significant residual movement at connections, or major settlement approach, vertical offset of the abutment, differential settlement at connections, shear key failure at abutments | Operational after repair | No |
| DS4 (complete) | Any column collapsing and connection losing all bearing support, which may lead to imminent deck collapse, tilting of substructure due to foundation failure | Not repairable | No |

Table 17. Earthquake damage states for railway bridges

#### 3.1.4.2 Key metric for consideration

The intensity threshold that damages the component (e.g., bridge piers) is assumed to be Modified Mercalli Intensity (MMI) VII–VIII, which is equivalent to a PGA of 0.3g (FEMA 2013a, USGS seismic, & JMA seismic). A damage state that is higher than DS3 (i.e., more than extensive/severe damage) is adopted to estimate the resiliency index, such as damage probability. In addition, as an example of the recovery process, Figure 50 shows a timeline of restoration for railway bridges after an earthquake (FEMA 2013a).

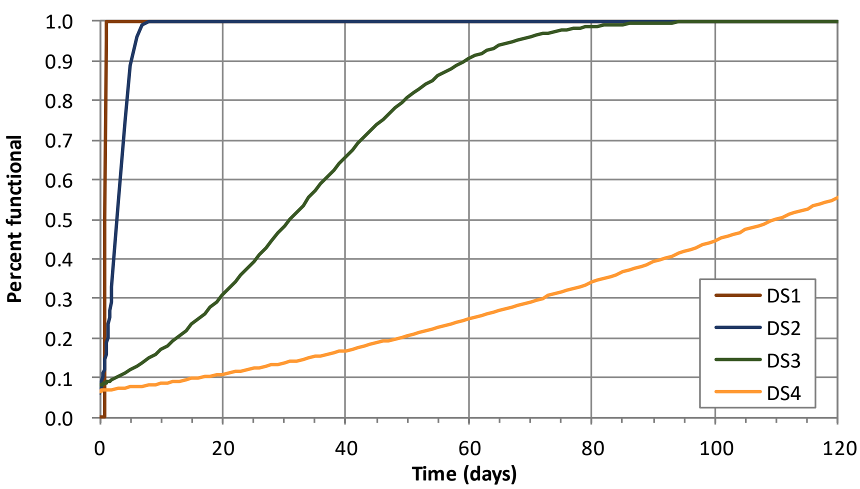



Figure 50. Restoration curve, railway bridge, earthquake hazard

#### 3.1.4.3 Infrastructure improvements

For railway systems, earthquake performance can be improved by several methods, including the methods in the following bullet list. For the most critical component, such as a bridge pier, column/pier jacketing with concrete or steel, as mentioned in the first bullet, improves axial, shear, and flexural capacity, and it is an effective engineering improvement for exiting bridge piers (Caltrans 2008). Quality assurance (QA) typically consists of testing and inspection of construction. Based on literature from the U.S. Department of Transportation (FHWA 2016), it is presumed that the standard-level QA uses about 10% of the project funds, and the higher-level QA takes approximately 30% of the total budget. Therefore, for better QA of retrofit work, the higher-level QA approach is considered in this report, as shown in the last bullet, and the standard-level QA is assumed to have been applied to the existing components.

Improvements to increase the earthquake resistance of bridge piers include:

- Improve the seismic capacity of bridge piers with concrete/steel jacketing; see Figure 51.
- Assess geotechnical components (e.g., tunnels and roadbeds) and strengthen them if any seismic deficiency exists.
- Ensure that buildings (part of railway systems) meet seismic code requirements and retrofit them as needed.
- Properly anchor equipment, including in fuel facilities, in DC substations, etc.
- Use seismic protection devices to reduce the load demand on the components or the buildings.
- Ensure that a seismic switch is installed to allow safe stop and safe restart of railway operations.
- Perform routine and regular maintenance for all components and fix any observed problems.
- Conduct the higher-level QA protocol (e.g., daily testing and continual and detailed field inspection).

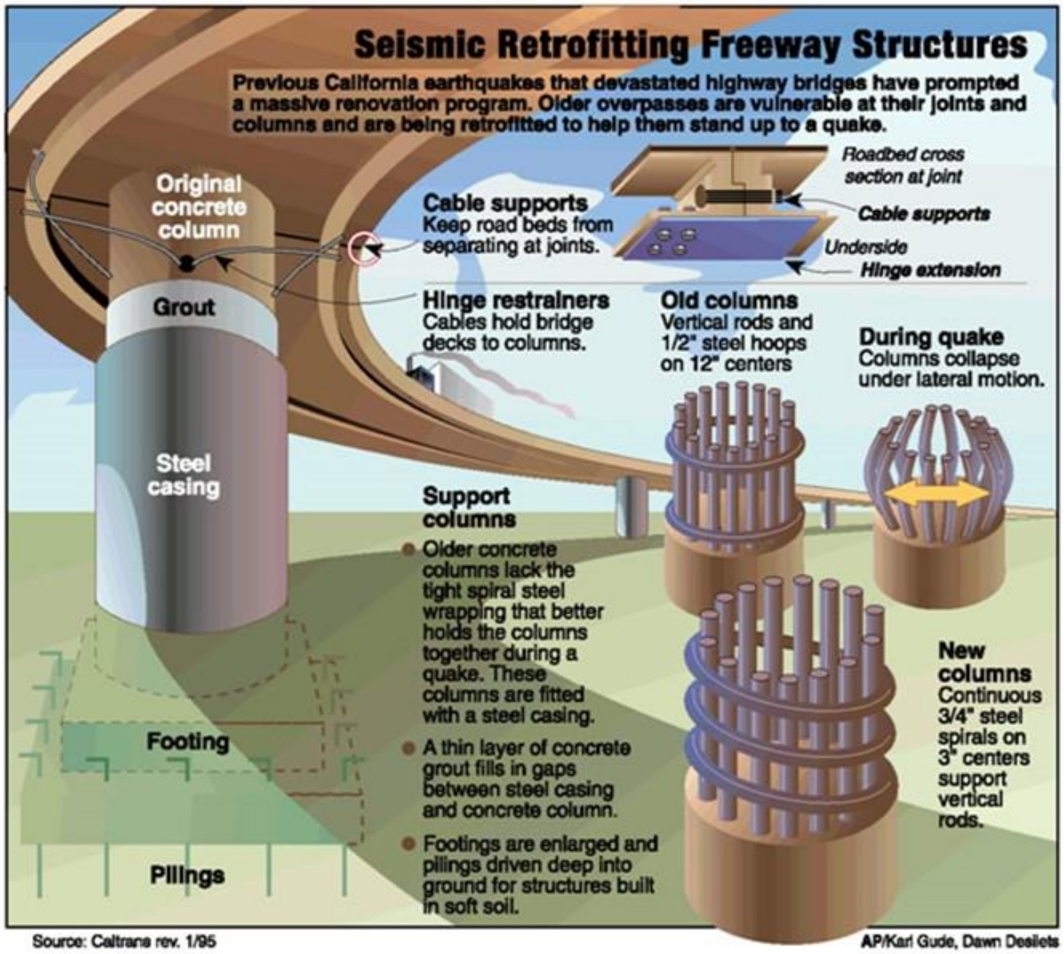
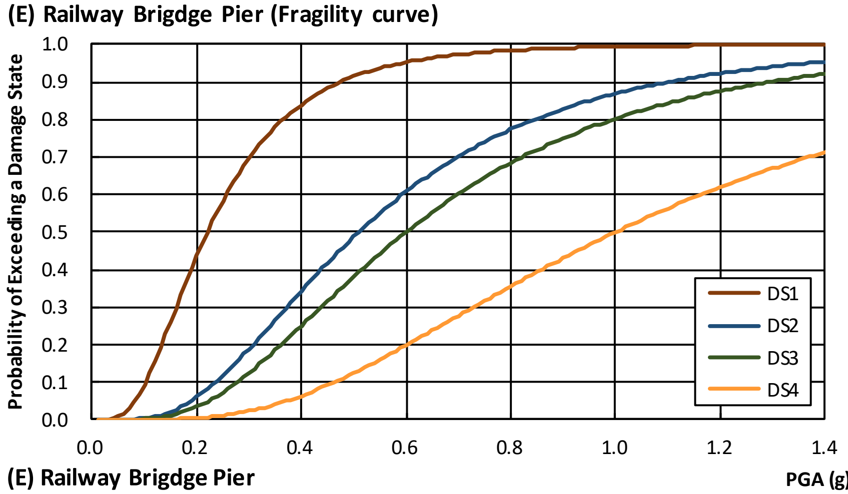



Figure 51. Example of bridge pier jacketing

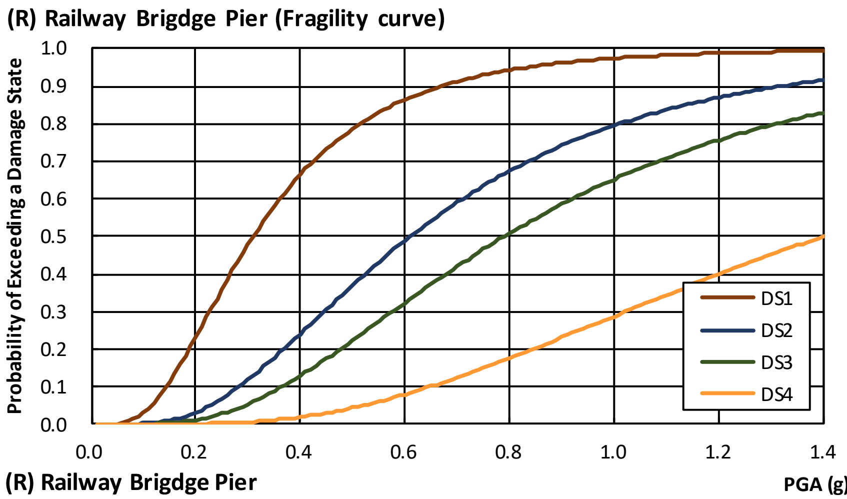
### 3.1.4.4 Cost-benefit considerations

FEMA (2013a) provides seismic fragility functions for railway bridges in use in the United States. Two sets of fragility functions are provided: one for railway bridges that are designed conventionally and one for railway bridges that have an appropriate seismic design and for which the bridge pier design meets the requirements of modern seismic codes. In this report, the fragility parameters of these functions are used to estimate the damage probability (i.e., resiliency index) for railway bridges from seismic motion, and it is assumed that this resiliency index represents similar indices for bridge piers throughout the world.

Figure 52 presents the fragility functions for bridge piers before and after improvement, respectively. For an earthquake with a PGA of 0.3g (a value expected from an earthquake of MMI VII–VIII), the probability of exceeding DS3 is 12% for existing bridge piers and is 5% for improved bridge piers. In this report, the cost to improve this pier's seismic capacity (i.e., pier jacketing) is estimated at approximately 20% of the pier replacement cost (Caltrans 2008). The total improvement cost, including the higher-level QA, can then be estimated as 25% of the total cost of the pier construction. Because the damage probability is reduced by implementation of the improvement measure, the damage level of the pier will likely be reduced in any earthquake. This outcome also means that the restoration period will be shortened; faster restoration provides significant transportation benefits and helps in the community's recovery.




Existing bridge pier




Retrofitted bridge pier

Figure 52. Seismic fragility functions for bridge piers of railway systems

### *3.1.5 Liquefaction hazard*

#### 3.1.5.1 General

Tracks/roadbeds are damaged primarily by permanent ground deformation (PGD) such as horizontally forced displacement and lateral spreading of soil caused by earthquake-induced liquefaction (FEMA 2013a). Tracks/roadbeds that are laid in liquefiable and weak soils in zones of high seismicity are especially vulnerable to such liquefaction damage.

#### 3.1.5.2 Key metric for consideration

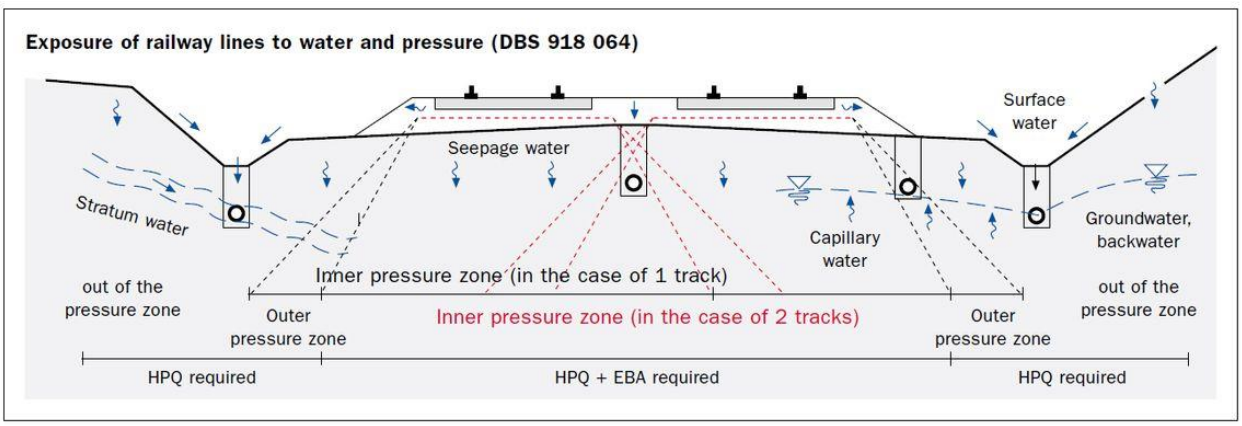
The intensity threshold that damages the component (i.e., tracks/roadbeds) is assumed to be a PGD of 12 in. (FEMA 2013a, Ferritto 1997, Urayasu 2012, & Nakashima et al. 2015), and damage states higher than DS3 (i.e., more than extensive/severe damage) are adopted to represent the resiliency index, such as damage probability. Liquefaction can damage railway systems and result in loss of operation of railway service. Also, because the failure is below ground, the cost of geotechnical repair and restoration of operations is relatively high.

### 3.1.5.3 Infrastructure improvements

For railway systems, liquefaction resistance can be improved by several methods, including the methods in the following bullet list. For the most critical components, such as tracks/roadbeds, installation of drainage and drainpipes, as proposed in the first bullet, improves the component strength to resist earthquake liquefaction, and it is an effective engineering improvement for existing tracks/roadbeds (Iowa DOT 2014). This method basically eliminates excess water from ballast pockets and saturated embankments. As stated in Section 3.1.4.3, it is assumed that the standard-level QA uses about 10% of project funds, and the higher-level QA takes approximately 30% of the total budget (FHWA 2016). Therefore, for better QA of retrofit work, the higher-level QA approach is considered in this report, as shown in the last bullet, and the standard-level QA is assumed to have been applied to the existing components.

Improvements to increase track/roadbed resistance to liquefaction include:

- Install drainage and drainpipes to reduce the amount of water in the soil; see Figure 53.
- Apply soil improvements and densification by several measures (e.g. cement mixing and soil compaction).
- Use deep foundations, such as pile, wall pile, or piled raft.
- Conduct the higher-level QA protocol (e.g., daily testing and continual and detailed field inspection).



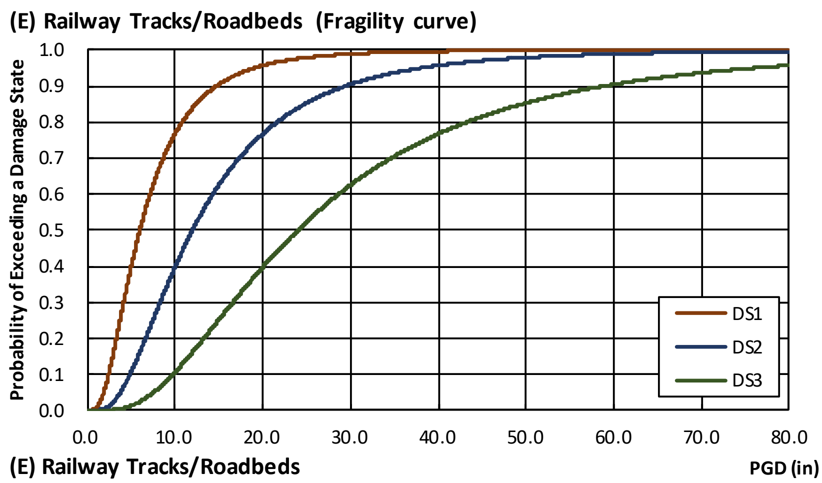

*Drainage systems in rail zones require excellent hydraulic and mechanical properties*

Figure 53. Example of drainage and drainpipe installation

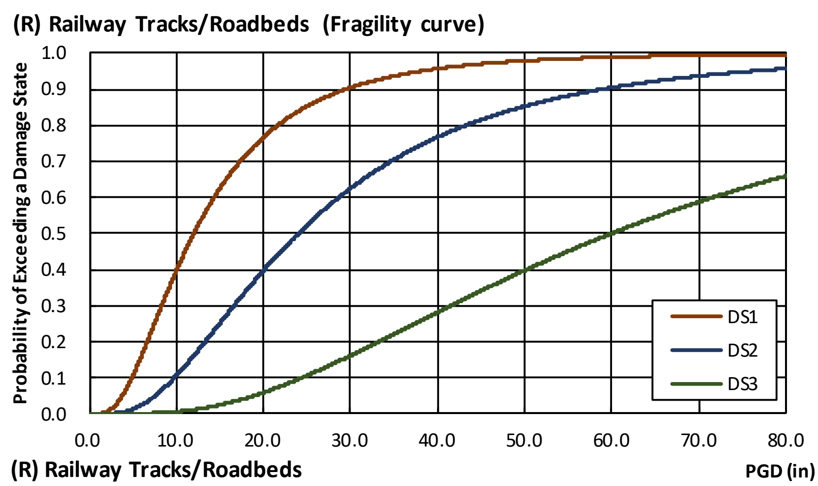
### 3.1.5.4 Cost-benefit considerations

FEMA (2013a) provides seismic fragility functions for railway tracks/roadbeds that are constructed in the United States. Two sets of fragility functions are provided: one for tracks/roadbeds that are designed normally and one for tracks/roadbeds that have higher performance. In this report, the difference between these fragility functions is assumed equivalent to the improvement that results from liquefaction strengthening. The fragility parameters of these functions are then applied to estimate the damage probability (i.e., resiliency index) for tracks/roadbeds from seismic liquefaction, and it is assumed that this resiliency index represents similar indices for tracks/roadbeds in other regions of the world.

Figure 54 shows the fragility functions for tracks/roadbeds before and after improvement, respectively. For earthquake liquefaction with a PGD of 12 in., the probability of exceeding DS3 is 16% for existing tracks/roadbeds and is 1% for improved tracks/roadbeds. In this report, the cost to improve the track/roadbed resistance to earthquake liquefaction (i.e., drainage and drainpipe installation) is estimated at approximately 35% of the track/roadbed replacement cost (Iowa DOT 2014). The total improvement cost, including the higher-level QA, can then be estimated as 45% of the total cost of the track/roadbed construction. Because the damage probability is significantly reduced by implementation of the improvement measure, it can be assumed that damage levels to tracks/roadbeds will be lower if liquefaction occurs. Less damage means that railway system operation can be restored more quickly.




Existing track/roadbed




Retrofitted track/roadbed

Figure 54. Ground displacement fragility functions for track/roadbeds of railway systems

### 3.1.6 *Wind hazard*

#### 3.1.6.1 General

The roofs and walls of railway stations are susceptible to damage from wind load, caused mainly by the peak gust wind speed (PGWS) from a hurricane or a typhoon (FEMA 2013b). The wind damage could be due to direct wind or crosswind (such as vortex shedding).

#### 3.1.6.2 Key metric for consideration

The intensity threshold that damages the component (i.e., railway station buildings) is assumed to be a PGWS of 90 mph, or 40 to 50 m/sec. (FEMA 2013b, NSSL, Beaufort Number, & JMA wind), and a damage state higher than DS3 (i.e., more than extensive/severe damage) is adopted to represent the resiliency index, such as damage probability. Strong wind can damage railway station buildings and result in loss of occupancy of railway stations. Also, train service would likely be inoperable because of impediments on the tracks or because of debris in or around damaged stations. The work that is required to repair and to restore wind-damaged stations is relatively minimal.

#### 3.1.6.3 Infrastructure improvements

For railway systems, wind resistance can be improved by several methods, including the methods in the following bullet list. For the most critical component, such as railway station buildings, retrofit of the wall-roof connection and replacement of the building envelope, as itemized in the first bullet, can improve the component wind capacity and are effectual engineering improvements for existing railway stations (FEMA 2007 & FEMA 2010). These retrofits are to strengthen the roof-wall connection and to update the windows, doors, and wall coverings to resist the current design wind load. As for earthquake and liquefaction improvements, it is similarly assumed that the standard-level QA uses about 10% of the project funds, and the higher-level QA takes approximately 30% of the total budget (FHWA 2016). Therefore, for better QA of retrofit work, the higher-level QA approach is considered in this report, as shown in the last bullet, and the standard-level QA is assumed to have been applied to the existing components.

Improvements to increase the wind resistance of railway stations include:

- Replace building envelopes and retrofit roof-wall connections; see Figure 55 and Figure 56.
- Strengthen the building-type components for a higher design wind speed and load.
- Ensure that all equipment (e.g., in the fuel facility and in the DC substation) is properly anchored by using positive mechanical attachments that are wind-rated.
- Confirm the wind resistance of railway bridges and piers and retrofit them if necessary.
- Conduct the higher-level QA protocol (e.g., daily testing and continual and detailed field inspections).

Accordion shutters for windows

Robust door/shutter

Figure 55. Example of building envelope improvement

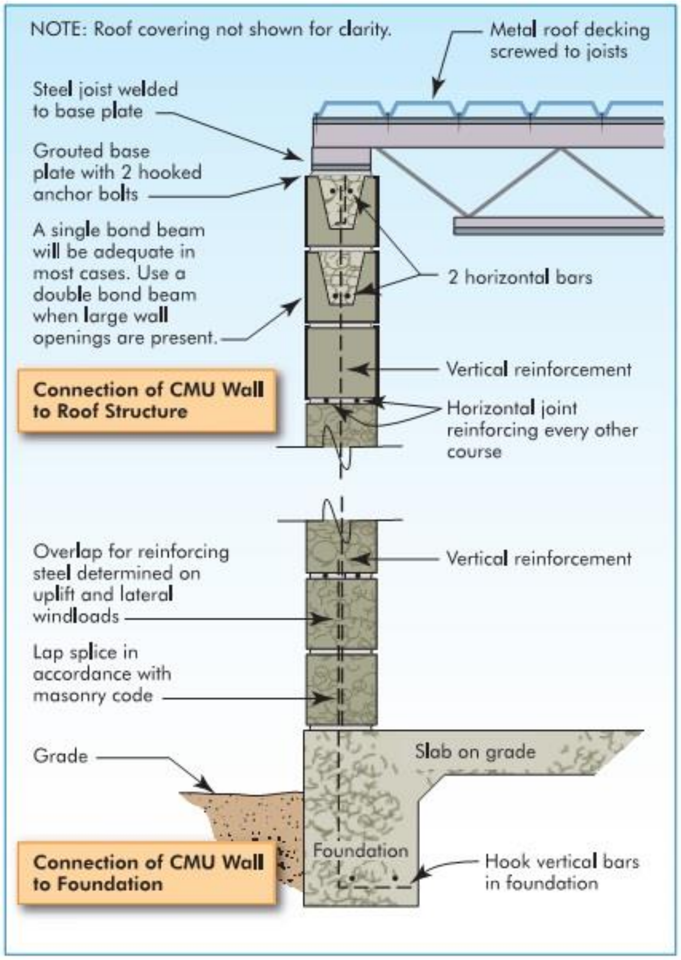
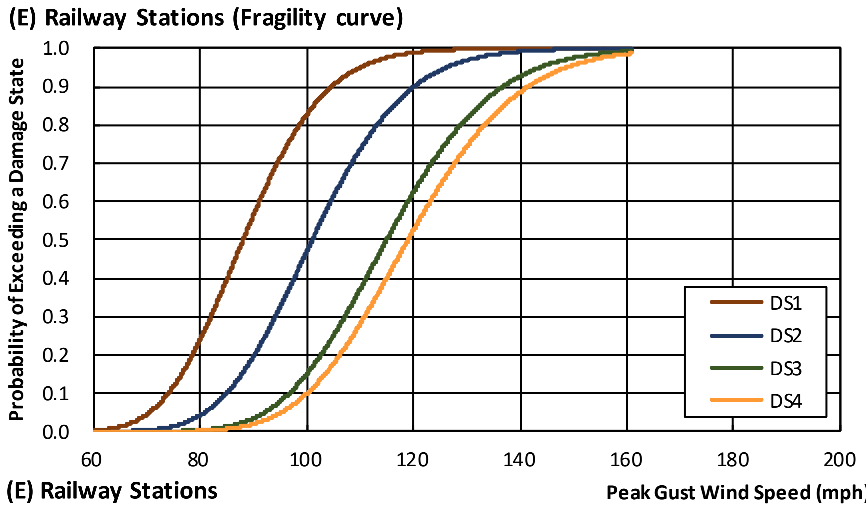



Figure 56. Example of appropriate roof-wall connection to resist wind load

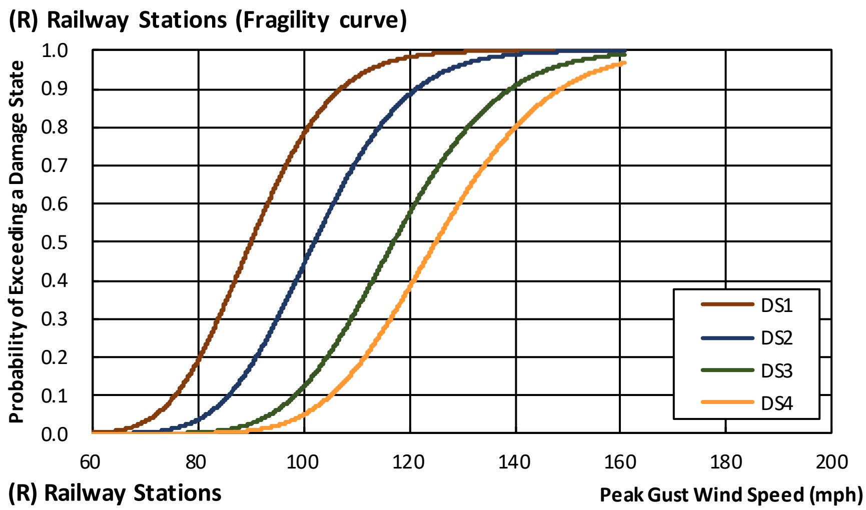
#### 3.1.6.4 Cost-benefit considerations

FEMA (2013b) provides wind fragility functions for buildings that are similar to railway stations that are constructed in the United States. Two sets of fragility functions are provided: one for buildings that have low-capacity roof-wall connections and one for buildings that have stronger, more robust roof-wall connections. In this report, the difference between these fragility functions is considered to be equivalent to the improvement that results from wind strengthening. The fragility parameters of these functions are then adapted to estimate the damage probability (i.e., resiliency index) for railway stations from wind hazard, and it is assumed that this resiliency represents similar indices of railway stations around the world.

Figure 57 illustrates the fragility functions for railway stations before and after improvement, respectively. For a strong wind with a PGWS of 90 mph, the probability of exceeding DS3 is 3.5% for existing stations and is 2.6% for improved railway stations. In this report, the cost to improve railway station resistance to wind hazard (i.e., wall-roof connection retrofit and building envelope replacement) is estimated at approximately 12% of the railway station replacement cost (FEMA 2010). The total improvement cost, including the higher-level QA, can then be estimated as 15% of the total cost of the railway station construction. The damage probability is reduced by implementation of the improvement measures. Less damage to stations will result in faster recovery and a more efficient restart of train operations after a wind disaster.




Existing railway station




Retrofitted railway station

Figure 57. Wind fragility functions for railway system stations

### *3.1.7 Flood hazard*

#### 3.1.7.1 General

The extent of flood damage at fuel/DC substations is greatly affected by the flood inundation depth (FID) that severe flooding causes. Flooding can damage emergency generators in addition to the substations, which can then create equipment bottlenecks that lead to cascading loss of railway operation (FEMA 2013c).

#### 3.1.7.2 Key metric for consideration

The intensity threshold that damages the component (i.e., fuel/DC substations) is assumed to be an FID of 3.3 ft., or 1 m (FEMA 2013c, U.S. Army Corps 2017, Huizinga et al. 2014, & MLIT 2014). The damage state is assumed to be either damaged (non-operable) or undamaged (operable), and the damage state is adopted to represent the resiliency index, such as damage probability. Electrical and mechanical equipment are primarily the components that are most vulnerable to a flood disaster, and malfunction of this equipment directly causes railway system outages (NYT 2012 & NYC 2013).

### 3.1.7.3 Infrastructure improvements

For railway systems, the capability to resist flooding can be improved by several methods, including the methods in the following bullet list. For the most critical component, such as fuel/DC substations, equipment elevation and watertight barrier installation, as listed in the first bullet, can improve the component flood resistance and are effective engineering improvements for existing fuel/DC substations (FEMA 2013d, FEMA 1999, & FEMA 2007). These improvements involve moving the substations and equipment to a higher location or building a pedestal and installing an enclosing wall to form a watertight barrier. As proposed in other hazard improvements in this report, the standard-level QA is assumed to use about 10% of the project funds, and the higher-level QA costs take approximately 30% of the total budget (FHWA 2016). Therefore, for better QA of retrofit work, the higher-level QA approach is considered in this report, as shown in the last bullet, and the standard-level QA is assumed to have been applied to the existing components.

Improvements to increase the flood resistance of railway systems include:

- Elevate the components (e.g., equipment) and install watertight barriers; see Figure 58.
- Install flood-monitoring sensors to notify operators when flooding occurs and as it reaches certain water levels for each component.
- Relocate critical components and equipment to a flood-safe location (i.e., a higher position).
- Duplicate the path of critical components and equipment to provide disaster redundancy.
- Estimate the potential inundation depth at the building’s location and prepare suitable countermeasures.
- Conduct the higher-level QA protocol (e.g., daily testing and continual and detailed field inspection).

Equipment elevation

Watertight barrier

Figure 58. Example of equipment elevation and watertight barrier

### 3.1.7.4 Cost-benefit considerations

A fragility function for inundation depth was developed based on actual damage records (Koshimura et al. 2009), and Figure 59 shows its empirical fragility curve, expressing the probability of exceeding a damage state and the inundation depth. In this report, it is assumed that this fragility function can be applied to large equipment such as in fuel/DC substations. Based on the proposed improvement methods in Section 3.1.7.3, the possible inundation depth can be greatly reduced. Therefore, for the railway system flood hazard in this report, it is assumed that engineering improvements will lower the inundation depth from an FID of 3.3 ft. for existing railway system components to an FID of 0.5 ft. for upgraded components.

The fragility parameters of the function are then applied to estimate the damage probability (i.e., resiliency index) for fuel/DC substations from flooding, and this resiliency index is assumed to represent similar indices of fuel/DC substations worldwide. For the FID of 3.3 ft. before any improvements, the probability of exceeding the non-operable damage state is 3.1%, and the FID of 0.5 ft. after implementation of improvements results in the probability of exceedance of 0.5% for fuel/DC substations.

In this report, the cost to improve the flood hazard resistance (i.e., equipment elevation and watertight barrier installation) is estimated at approximately 40% of the fuel/DC substation replacement cost (FEMA 1999). The total improvement cost, including the higher-level QA, can then be estimated as 50% of the total cost of the fuel/DC substation construction. The damage probability (i.e., resiliency index) is greatly reduced after the improvements have been implemented. Less fuel/DC substation damage means fewer railway system malfunctions and that operation can be restored in a shorter time. Such expedited recovery helps avoid the considerable loss of economic and public benefits.

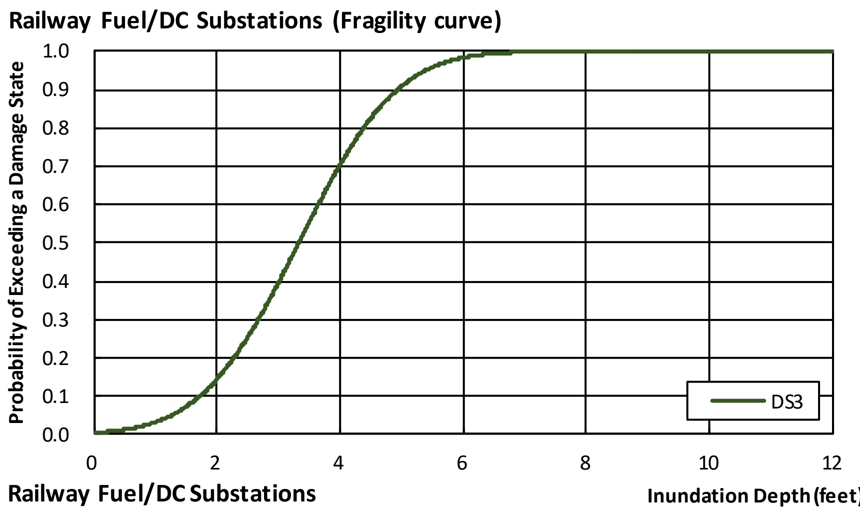



Figure 59. Inundation fragility functions for fuel/DC substations of railway systems

## 4. Water infrastructure

## 4.1 (Impounding/storage) reservoirs

### *4.1.1 Overview*

A water reservoir (from the French term *réservoir*, a “tank”) is a storage space for a large volume of water. Most water distribution systems use various types of reservoirs, including either tanks or open-cut reservoirs. Tanks (elevated or ground supported) can be constructed of steel, wood, or concrete. An “open-cut reservoir” usually means an enlarged natural or artificial lake, a storage pond, or an impoundment that is created by using a dam or a lock to store water. Reservoirs can be created by controlling a stream that drains an existing body of water. They can also be constructed in river valleys by using a dam. Alternatively, a reservoir can be built by excavating flat ground or by constructing retaining walls and levees. Figure 60 shows an example of a water reservoir.

Figure 60. Open-cut reservoir (San Pablo, California, USA)

### *4.1.2 Summary*

The results from a literature review of water reservoirs are summarized in Table 18and are presented in the “*Improvement of infrastructure resilience evaluation matrix for selected natural hazards*” table in the final report. The following subsections of this background report provide more detailed descriptions and background information.

| Hazard | Susceptible | Improvements | Damage index | | Approximate improvement cost as % of cost |
|---|---|---|---|---|---|
| | | | As-is | Improved | |
| **Earthquake** | Y | Maintenance, drenching, higher seismic design forces | 0.15 | 0.05 | 5% |
| **Liquefaction** | Y | Drive pre-stressed concrete piling | 0.2 | 0.02 | 20% |
| **Wind** | N | -- | -- | -- | -- |
| **Flood** | Y | Higher freeboard, drenching | 0.2 | 0.05 | 5% |

Table 18. Summary of findings for water reservoirs

### *4.1.3 Vulnerability to natural hazards*

For seismic considerations, open-cut reservoirs are considered embankment dams. Water reservoirs have been significantly damaged in past natural hazard events. Figure 61presents examples.

Earthquake (Fujinuma Dam, 2011 earthquake)

Liquefaction (1971 San Fernando Earthquake)

This hazard does not apply.

Wind

Flooding (Teton Dam, Idaho, USA, 1976)

Figure 61. Damage to water reservoirs from natural hazards

### *4.1.4 Earthquake hazard*

#### 4.1.4.1 General

Water reservoirs are susceptible to damage from earthquakes. Modes of failure include:

- Sliding and cracking—concrete dams
- Liquefaction of foundation (discussed in Section 4.1.5)
- Embankment deformation and loss of freeboard
- Cracking of the embankment, leading to piping
- Fault displacement of a dam foundation
- Overtopping from landslides into the reservoir

For the failure mode of cracking and crest settlement, Pells and Fell (2003) developed a number of damage states, as listed in Table 19

| **Damage state** | **Description** | **Longitudinal crack width, mm** | **Crest settlement/height, %** |
|---|---|---|---|
| DS0 | None/slight | <10 | <0.03% |
| DS1 | Minor | <30 | <0.2% |
| DS2 | Moderate | <80 | <0.5% |
| DS3 | Major | <150 | <1.5% |
| DS4 | Severe | <500 | <5% |
| DS5 | Collapse | >500 | >5% |

Table 19. Earthquake damage states for water reservoirs, internal erosion through cracking and crest settlement

#### 4.1.4.2 Key metric for consideration

The key parameters for open-cut reservoirs are the loss of contents and preservation of the structural integrity of the dam. Swaisgood (2014) provides an estimate of normalized (with respect to the structure height) crest settlement as a function of peak ground acceleration (PGA) and earthquake magnitude. Figure 62shows data for a design-level earthquake of $M_w$ 6. Note that for a PGA of 0.6g, the normalized settlement of 0.4% is expected, which falls in the moderate damage state (DS2) in Table 19. When the same reservoir is subjected to a $M_w$ 8 earthquake with the same PGA of 0.6g, the normalized settlement would be 1%, and it would be classed as the major damage state (DS3) in Table 19. . The larger crest settlement could initiate overtopping, piping, and failure of the reservoir.

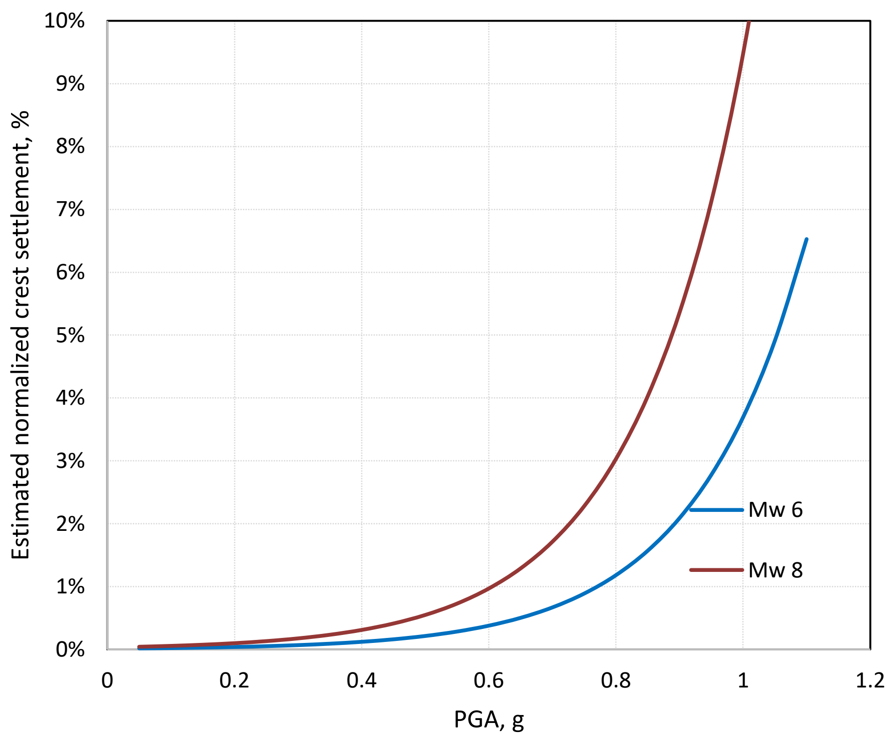



Figure 62. Normalized crest settlement, water reservoirs, earthquake hazard

#### 4.1.4.3 Infrastructure improvements

For water reservoirs, earthquake performance can be improved by several methods. Examples include:

- Consider a higher operational design and basic safety earthquake return intervals (larger earthquakes).
- Drenching.
- Perform routine and periodic maintenance of the reservoir and fix any problems.
- For new infrastructure, use seismic design provisions, and for existing assets, perform seismic retrofitting.

#### 4.1.4.4 Cost-benefit considerations

For reinforced-concrete dams, see the section 2.2 that discusses hydroelectric power plants and dams. For open-cut embankment (earth) reservoirs, the additional cost for improvements is estimated as 5% of replacement cost, and the improvements are assumed to reduce from 15% of damage probability for the existing condition to 5% for the improved state.

### *4.1.5 Liquefaction hazard*

#### 4.1.5.1 General

Earth dams are typically constructed in areas that are susceptible to liquefaction. The increase in pore water pressure due to liquefaction reduces the soil's shear strength. Reservoir loading exceeds the shearing resistance that remains in the layer, and the entire embankment slides downstream. The depth and velocity of water that flows through the gap can erode the materials along the sides and across the bottom of the gap. One type of erosion, known as "head cutting," carves channels across the crest. The channels can widen and deepen to a point at which the embankment is breached and water in the reservoir is released.

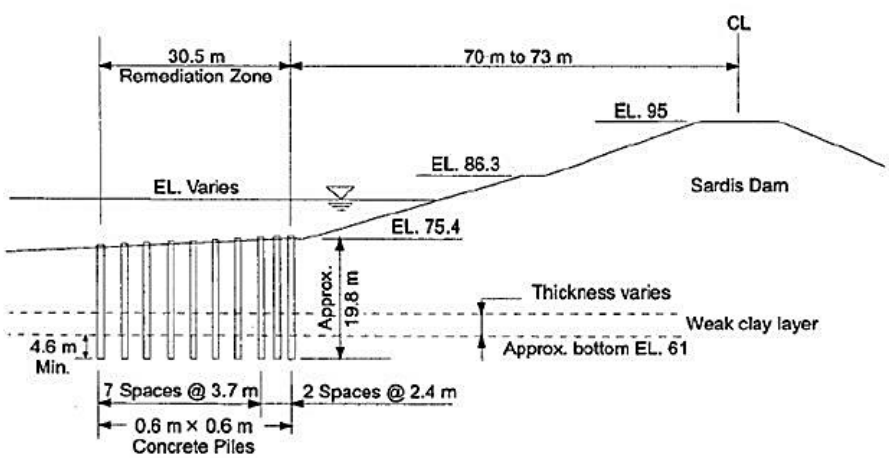
#### 4.1.5.2 Key metric for consideration

The first item to evaluate is the likelihood that a continuous layer or zone of potentially liquefiable material exists within the dam or the foundation. Soil property data from site testing can provide insights into the potential for such a continuous liquefiable layer. Data from a cone penetration test (CPT) that is readily available can be correlated to the cyclic stresses and to the plasticity index to help develop the likelihood of liquefaction.

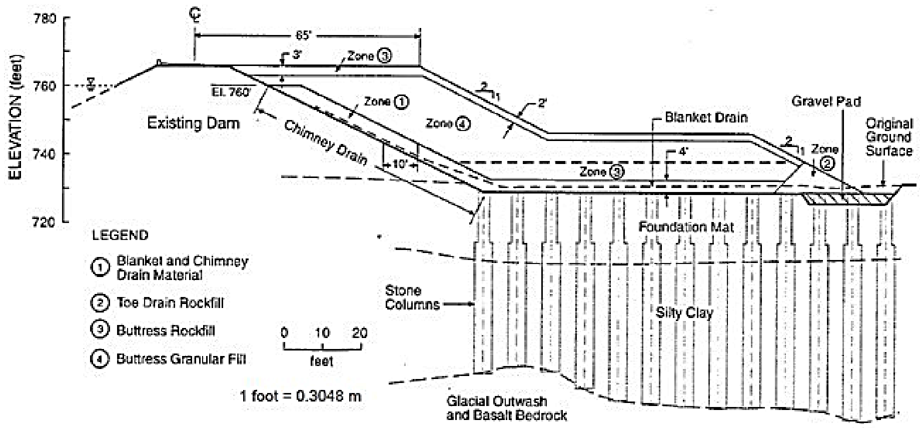
#### 4.1.5.3 Infrastructure improvements

Examples of improvements to help reservoirs resist liquefaction include the following (Mejia 2005); see Figure 63:

- Drive pre-stressed concrete piles below the liquefiable layer to stabilize the foundation and the abutments.
- Place a buttress on the downstream slope of the dam and strengthen the foundation beneath the buttress with stone columns.
- Ensure that the freeboard allows crest settlement; drains and filters can accommodate the expected shear deformations. The freeboard is the difference in elevation between the dam crest and the water surface in the reservoir.




Sardis Dam, Mississippi, USA (pre-stressed concrete piles)




South Tolt Dam, Washington, USA (buttress on the downstream slope)

Figure 63. Liquefaction mitigation for reservoirs

#### 4.1.5.4 Cost-benefit considerations

Retrofitting for liquefaction is generally a safety issue, not an economic issue. The potential for failure is assumed to be reduced from 20% for existing reservoirs to 2% for retrofitted reservoirs, and the increased cost is estimated as 20%.

### *4.1.6 Wind hazard*

#### 4.1.6.1 General

Water reservoirs are not adversely affected by windstorms, and because they are typically located away from shorelines, they are not affected by hurricanes.

#### 4.1.6.2 Key metric for consideration

This topic does not apply.

#### 4.1.6.3 Infrastructure improvements

This topic does not apply.

#### 4.1.6.4 Cost-benefit considerations

This topic does not apply.

### *4.1.7 Flood hazard*

#### 4.1.7.1 General

Water reservoirs are one way to provide flood protection downstream. However, the reservoirs themselves can be subject to flooding. Overtopping of the reservoir can occur when flooding causes upstream tributaries to discharge into the dam.

#### 4.1.7.2 Key metric for consideration

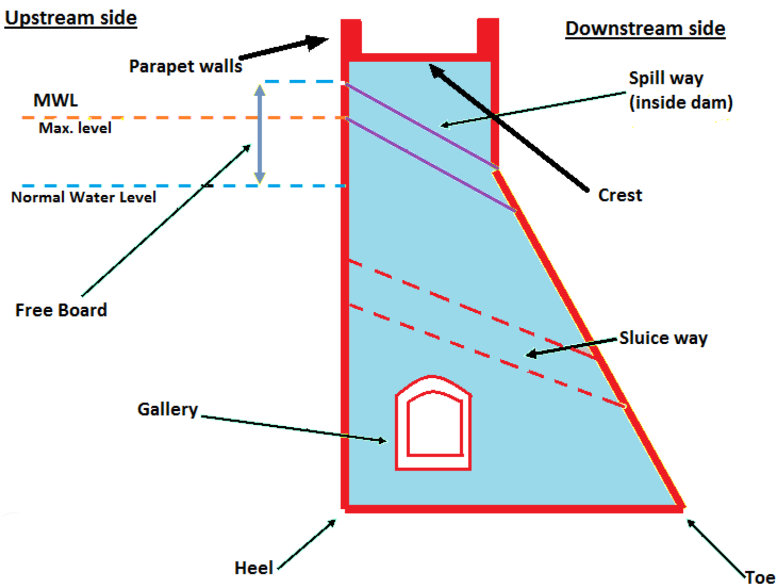
The key consideration is to avoid overtopping of the dam during extreme flooding.

#### 4.1.7.3 Infrastructure improvements

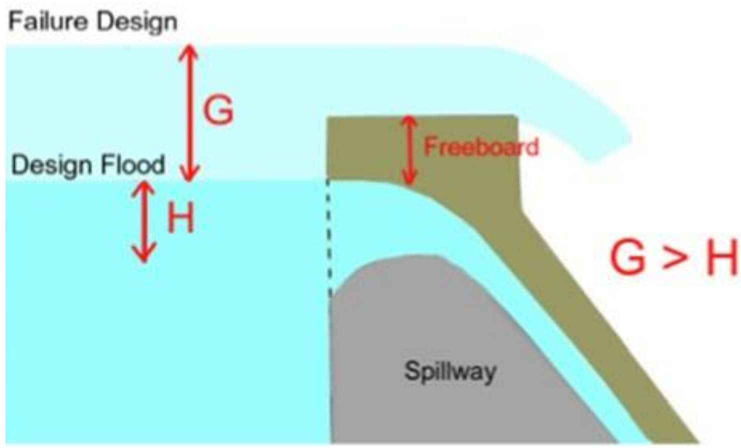
For water reservoirs, flood performance can be improved by several methods. Examples include:

- Ensure that the reservoir is properly drenched regularly to remove excess sediment and to reduce the chance of overtopping.

- Ensure that the dam has adequate spillway capacity and that there is remote access to operate the spillways.
- Increase the freeboard, defined as the difference in elevation between the dam crest and the water surface in the reservoir. As Figure 64 shows, overtopping failure occurs when the freeboard is insufficient.




Cross section of reservoir




Freeboard failure

Figure 64. Spillway rating curve (capacity) as a function of spillway height

### 4.1.7.4 Cost-benefit considerations

Typically, the freeboard is designed for a 100-year flood. In flood-prone areas, to reduce the probability of overtopping, a 1,000-year flood (the probable maximum flood) can be used for design of the freeboard.

As shown in Figure 65 (USDI 2012) for a hypothetical dam, additional freeboard significantly reduces the risk of failure from overtopping. For example, the addition of 0.5 ft. of freeboard reduced the probability of failure from overtopping by a factor of 10. In this report, improvement by adding 1 m of freeboard, or by drenching to achieve the same effect, is considered to reduce flood overtopping failure risk by a factor of 4. The increase in cost for the additional freeboard is only a small percentage of the total cost (FEMA 2008); it is assumed as 5% in this report.

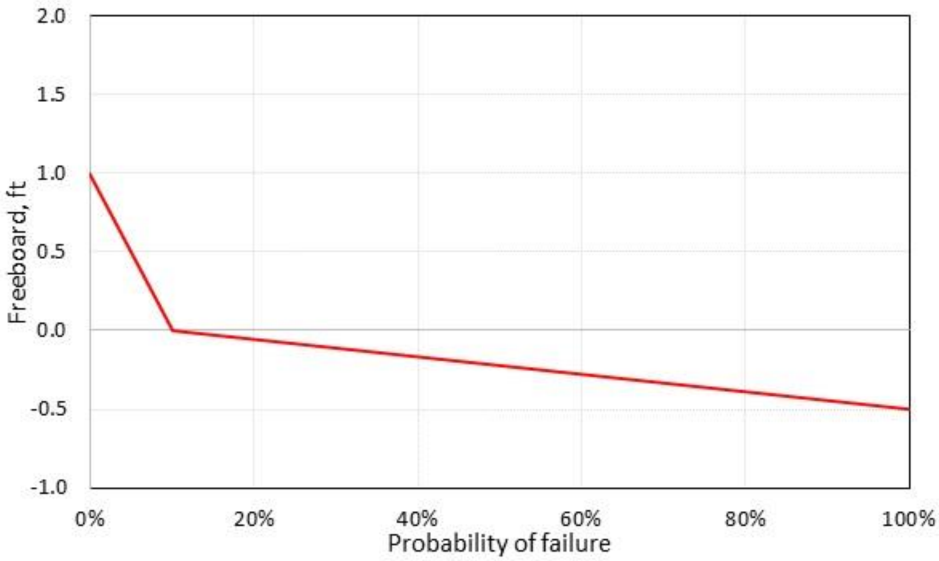



Figure 65. Example of overtopping fragility curve

## 4.2 Reservoirs (storage tanks)

### *4.2.1 Overview*

As previously described, a water reservoir is a storage space for a large volume of water. Most water distribution systems use various types of reservoirs, including either tanks or open-cut reservoirs. Tanks (elevated or ground supported) can be constructed of steel, wood, or concrete and are discussed further in this section. Figure 66 shows examples of water storage tanks. In this section, ground tanks are the focus of discussion.

Elevated

Ground supported

Figure 66. Water storage tanks

### *4.2.2 Summary*

The results from a literature review of water storage tanks are summarized in Table 20 and are presented in the "*Improvement of infrastructure resilience evaluation matrix for selected natural hazards*" table in the final report. The following subsections of this background report provide more detailed descriptions and background information.

| **Hazard** | **Type** | **Susceptible** | **Improvements** | **Damage index** | | **Approximate improvement cost as % of cost** |
|---|---|---|---|---|---|---|
| | | | | **As-is** | **Improved** | |
| **Earthquake** | Ground | Y | Proper anchorage and seismically designed components | 0.20 | 0.02 | 5% |
| | Elevated | Y | | | | |
| **Liquefaction** | Ground | Y | Deep foundations | 0.40 | 0.10 | 50% |
| | Elevated | N | | | | |
| **Wind** | Ground | N | Design for 250-km/hr. wind | 0.20 | 0.05 | 10% |
| | Elevated | Y | | | | |
| **Flood** | Ground | N | -- | -- | -- | -- |
| | Elevated | N | | | | |

Table 20. Summary of findings for water storage tanks

### *4.2.3 Vulnerability to natural hazards*

Water storage tanks have been significantly damaged in past natural hazard events. Figure 67 presents some examples.

Earthquake (Algeria, 1980 earthquake)

This hazard does not apply.

Liquefaction

Wind

This hazard does not apply.

Flood

Elevated tanks

Earthquake (California, 2010)

Liquefaction (2011 Christchurch Earthquake)

Ground-level (grade) tanks

Figure 67. Damage to water tanks from natural hazards

### *4.2.4 Earthquake hazard*

#### 4.2.4.1 General

Water storage tanks are susceptible to damage from peak ground acceleration (PGA). Modes of failure include:

- Cracking and spalling of concrete tanks
- “Elephant foot” buckling of ground-level steel tanks
- Shear failure of concrete tanks
- Failure of support members and connections for elevated tanks

- Anchorage failure

Damage states are defined as listed in Table 21 (FEMA 2013a).

| Damage state | Definition | Restoration, days (median) |
|---|---|---|
| DS0 (none) | No damage | -- |
| DS1 (minor) | Tank suffering minor damage without loss of its contents or functionality. Minor damage to the tank roof due to water sloshing, minor cracks in concrete tanks, or localized wrinkles in steel tanks fits the description of this damage state. | 1 |
| DS2 (moderate) | Tank being considerably damaged, but only minor loss of content. Elephant foot buckling for steel tanks without loss of content or moderate cracking of concrete tanks with minor loss of content fits the description of this damage state. | 3 |
| DS3 (extensive) | Tank being severely damaged and going out of service. "Elephant foot" buckling for steel tanks with loss of content, stretching of bars for wood tanks, or shearing of wall for concrete tanks fits the description of this damage state. | 93 |
| DS4 (complete) | Tank collapsing and losing all of its content. | 155 |

Table 21. Earthquake damage states for water storage tanks

### 4.2.4.2 Key metric for consideration

The time that it takes to restore damaged water storage tanks to full operation is a key metric that is considered in this report. Figure 68 presents the restoration timeline (FEMA 2013a) for water storage tanks.

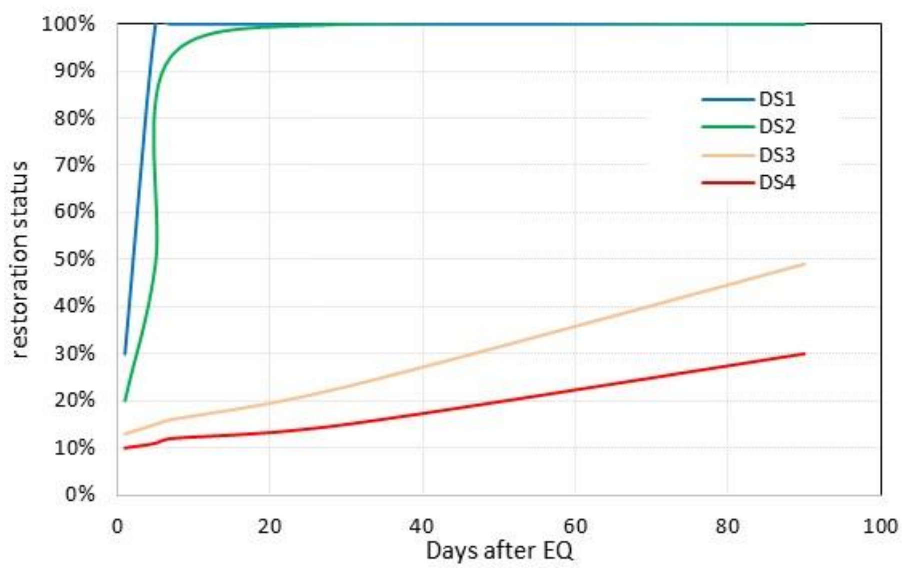



Figure 68. Restoration curve, water storage tanks, earthquake hazard

### 4.2.4.3 Infrastructure improvements

For water storage tanks, earthquake performance can be improved by several methods. Examples include:

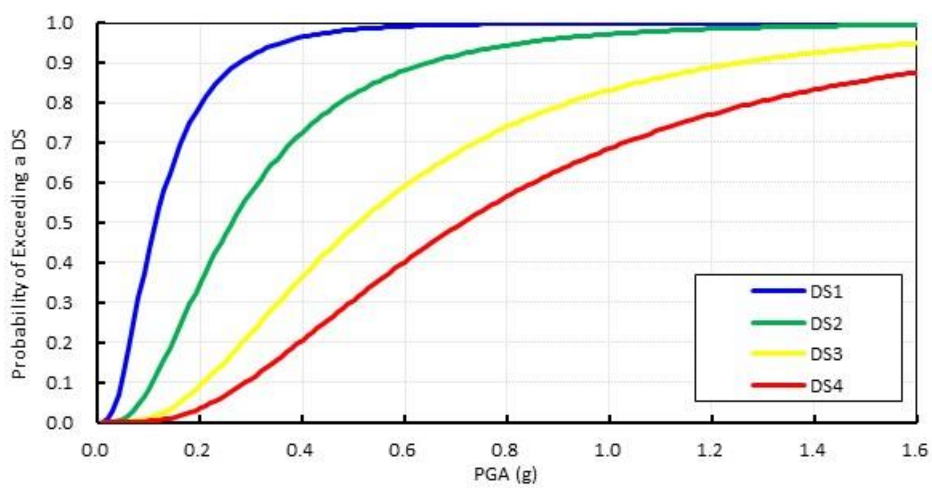
- Use a larger design earthquake for critical facilities.
- Ensure that the tank is seismically designed and that all members and connections are designed for seismic forces.

- For ground-level steel tanks, use stiffeners to delay the onset of local buckling.
- For concrete tanks, ensure that the walls are sufficiently reinforced to prevent shear failure.
- For elevated towers, design the members and connections to remain elastic under seismic loading.
- Ensure that proper types of foundation anchorage are used and that they have adequate embedment.

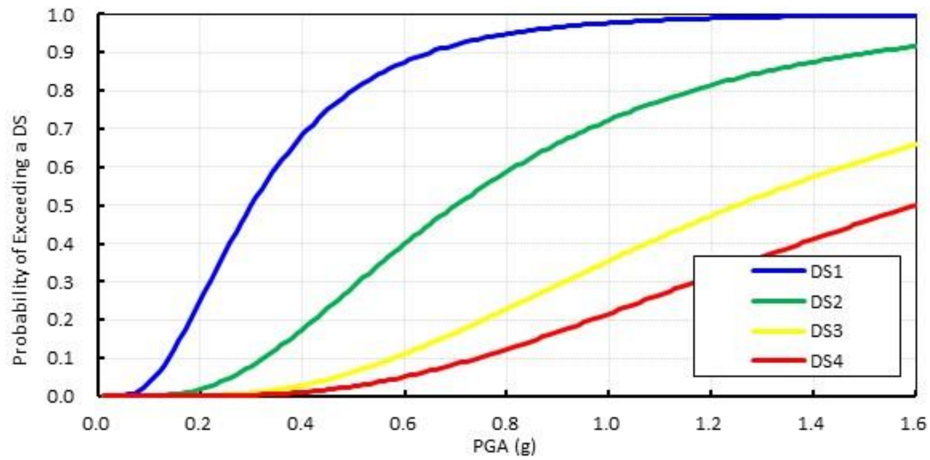
#### 4.2.4.4 Cost-benefit considerations

FEMA (2013a) provides fragility functions for water storage tanks in use in the United States. Two sets of data are provided: one for units with proper anchorage and one for unanchored tanks. In this report, the fragility parameters are modified for the unanchored cases to account for the higher variability and the lower expected quality in worldwide application, and the same ratio of unanchored-to-anchored mean values are used for the ground-level and elevated tanks. Figure 69 and Figure 70 present the fragility functions for the unanchored and anchored cases for ground and elevated water tanks, respectively. Note that for an earthquake with a PGA of 0.4g (a value that is expected from an $M_w$ 7 earthquake):

- For ground-level tanks, the probability of exceeding DS4 is 20% for unanchored tanks and is 2% for anchored tanks. In other words, by using seismic design and detailing (estimated as approximately a 5% cost increase in this report), tanks will likely survive the earthquake and will be operational within days.
- For elevated tanks, the probability of exceeding DS4 is 18% for unanchored tanks and is 1% for anchored tanks. In other words, by using seismic design and detailing (estimated as approximately a 5% cost increase in this report), tanks will likely survive the earthquake and will be operational within days.

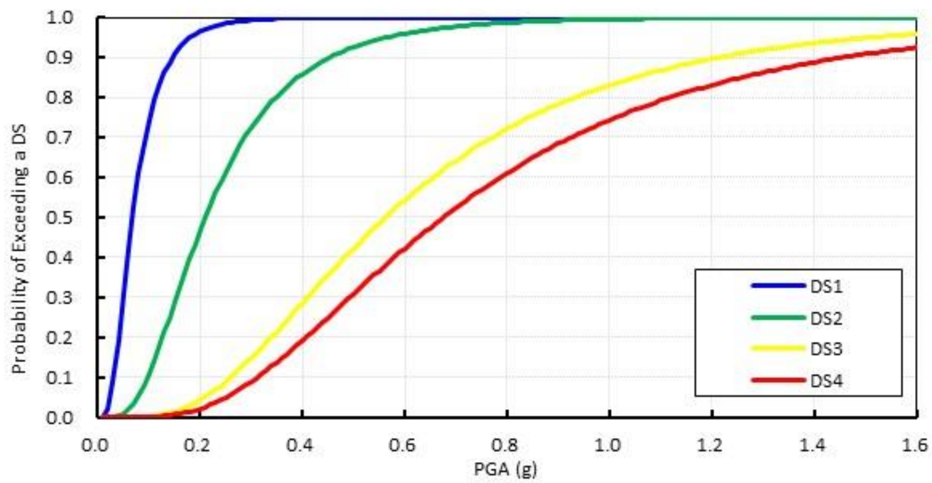



Unanchored subcomponents

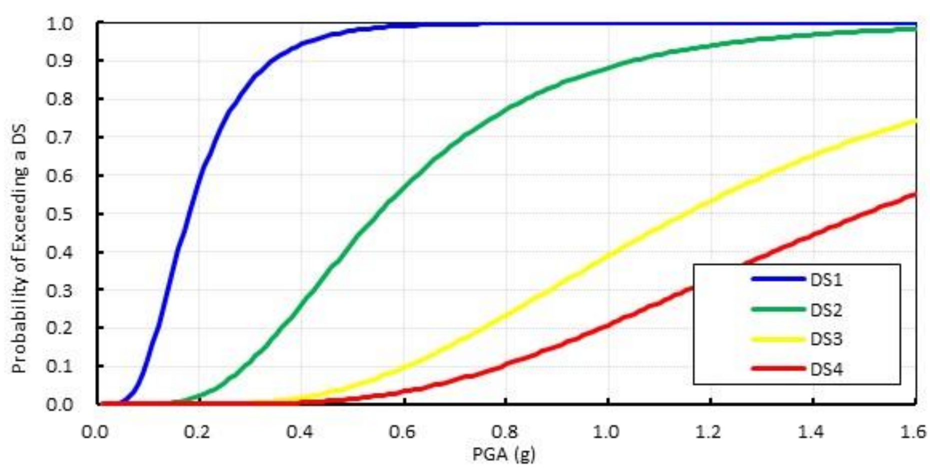



Anchored and seismic subcomponents

Figure 69. Damage fragility functions for water storage tanks, ground level




Unanchored subcomponents




Anchored and seismic subcomponents

Figure 70. Damage fragility functions for water storage tanks, elevated

### *4.2.5 Liquefaction hazard*

#### 4.2.5.1 General

Elevated water tanks are not constructed in liquefaction zones. Liquefaction can affect ground-level water storage tanks, however, so they are the focus of this section.

#### 4.2.5.2 Key metric for consideration

Ground displacement due to liquefaction is a key concern. Ground displacement can result in tilting or damage to storage tanks.

#### 4.2.5.3 Infrastructure improvements

Yoshizawa et al. (2000) studied the performance of three tanks in the aftermath of the 1995 Kobe Earthquake. The three tanks were 37 m in diameter and 37 m tall and were approximately 30 m apart. Table 22 summarizes the findings.

| Tank | Foundation | Settlement, mm | Inclination, % |
|---|---|---|---|
| A | Pile | 0 | 0 |
| B | Raft | 625 | 1.25% |
| C | Raft | 440 | ~0 |

Table 22. Performance of the three tanks

For tanks B and C, the soil was improved by using vibroflotation. Note that deep foundations are most effective for liquefaction mitigation. The conclusion of the study was that soil compaction to a greater depth is more effective than enlarging the width of the compaction area that is outside the tank footprint.

#### 4.2.5.4 Cost-benefit considerations

For new tanks in liquefiable zones, the use of concrete piles is recommended. To strengthen existing tanks that are in a liquefiable zone, secant piles can be used. Secant piles tend to homogenize settlement under a tank (Saez and Ledezma, 2014), and by reducing the differential settlement, they reduce damage from ground liquefaction. Figure 71 shows an example of secant piles. Piling can be expensive, and a reservoir owner should consider alternatives, such as relocating tanks. If relocation is not an option, then the anticipated additional cost is 50%, with an improvement estimated at 75%.

Figure 71. Secant piles

### *4.2.6 Wind hazard*

#### 4.2.6.1 General

Because their design is governed by seismic forces, ground-level tanks typically are not affected by wind loading except in large hurricanes. Elevated water tanks could fail due to wind forces, however. The areas of vulnerability for elevated tanks include:

- High stress that is induced in the tank itself
- The demand for the members and the connection of the supporting tower
- Because of their lighter weight, steel tanks are more vulnerable to damage from wind forces

#### 4.2.6.2 Key metric for consideration

The level of damage in steel tanks depends on the water elevation inside the tank (Tansel and Ahmed 2000) because the external pressure is higher than the internal pressure. Following are the main conclusions from Tansel and Ahmed (2000):

- During severe weather conditions such as hurricanes, try to maintain tanks at full capacity. Elevated water storage tanks are typically built to withstand winds of 150 mph (i.e., 240 km/hr), which would be exceeded during hurricanes. Empty or partially full water storage tanks can collapse at the top section.
- Perform regular maintenance and inspection for cracks and leaks.

#### 4.2.6.3 Infrastructure improvements

Recommended improvements for water storage tanks are to follow the National Fire Protection Association (NFPA) guidelines. The NFPA (2018) states:

> *Anchor bolts shall be arranged to securely engage a weight at least equal to the net uplift when the tank is empty and the wind is blowing from any direction. Lightweight tanks definitely need to be anchored against high winds in areas that experience them, and elevated water tanks should have their windage rods inspected and tightened regularly to maintain winds of 150 mph, blowing from any direction.*

#### 4.2.6.4 Cost-benefit considerations

Concrete tanks are more resistant to wind loading, so they can be considered as an effective option. For lightweight steel tanks, the tank itself, the support frame, and all the connections and foundation anchorage should be designed for a minimum wind speed of 250 km/hr. The additional material and labor costs for improvement to withstand this wind force are estimated at 10%. The resulting reduction in damage is estimated to be from 20% for tanks with a lower design wind force to 5% for tanks that are designed for at least 250 km/hr.

### *4.2.7 Flood hazard*

#### 4.2.7.1 General

Elevated water storage tanks are not vulnerable to flooding. However, ground-level water tanks can be damaged and lose capacity if the water level in the tank is below the floodwater level. Vulnerabilities are defined as listed in Table 23(FEMA 2013c).

| Type | Overall vulnerability | Inundation | Scour | Hydraulic pressure & debris | Financial loss | Assumptions |
|---|---|---|---|---|---|---|
| At grade | Low | Low | -- | -- | Low | The water level in the tank exceeds the flood depth (so the tank will not float) |
| Elevated | Low | Low | -- | -- | Low | Tank foundations are not damaged |

Table 23. Flood vulnerability, aboveground water tanks

#### 4.2.7.2 Key metric for consideration

The key consideration is to avoid low water levels for at-grade tanks and to ensure that the foundation is anchored for elevated tanks.

#### 4.2.7.3 Infrastructure improvements

This topic does not apply.

#### 4.2.7.4 Cost-benefit considerations

This topic does not apply.

## 4.3 Water and wastewater treatment plants

### *4.3.1 Overview*

Water treatment plants (WTPs) improve the quality of water to make it more acceptable for a specific end use. The end use might be for consumption, industrial water supply, irrigation, river flow maintenance, water recreation, etc., or it might be safely returned to the environment. WTPs remove contaminants and undesirable components, or they reduce the concentration of these elements enough so that the water is satisfactory for its intended end use.

Wastewater treatment is a process that converts wastewater into an effluent that can be returned to the water cycle with minimal impact on the environment or that can be directly reused. The wastewater treatment process takes place in a wastewater treatment plant (WWTP), often referred to as a water resource recovery facility (WRRF), or in a sewage treatment plant. Pollutants in municipal wastewater, which includes wastewater from households and from small industries, are removed or are broken down into less harmful components. Figure 72 shows an example of a water reservoir within a WWTP. A wastewater system typically consists of collection sewers, interceptors, lift stations, and WWTPs.

WTPs and WWTPs have similar levels of vulnerability (FEMA 2013a); therefore, they are discussed together in this section of the report. As a group, they are referred as WTP/WWTPs.

Figure 72. Wastewater treatment plant, Massachusetts, USA

### *4.3.2 Summary*

The results from a literature review of WTP/WWTPs are summarized in Table 24 and are presented in the "*Improvement of infrastructure resilience evaluation matrix for selected natural hazards*" table of the project final report. The following subsections of this background report provide more detailed descriptions and background information.

| Hazard | Susceptible | Improvements | Damage index | | Approximate improvement cost as % of cost |
|---|---|---|---|---|---|
| | | | As-is | Improved | |
| **Earthquake** | Y | Higher threshold seismic design | 0.7 | 0.4 | 15% |
| **Liquefaction** | Y | Higher threshold for permanent ground displacement | 0.7 | 0.4 | 20% |
| **Wind** | N | -- | -- | -- | -- |
| **Flood** | Y | Elevation of equipment<br>Floodproofing | 0.5<br>0.5 | 0.2<br>0.2 | 5%<br>5% |

Table 24. Summary of findings for WTP/WWTPs

### *4.3.3 Vulnerability to natural hazards*

WTP/WWTPs have been significantly damaged by natural hazard events in the past. Figure 73 shows examples.

Earthquake (Japan)

Liquefaction, wastewater treatment plant (New Zealand)

Liquefaction, sewage manhole (Japan)

Flooding (United States)

Figure 73. Damage to WTP/WWTPs from natural hazards

### *4.3.4 Earthquake hazard*

#### 4.3.4.1 General

WTP/WWTPs are susceptible to damage from peak ground acceleration (PGA) that is caused by an earthquake. Modes of failure can include:

- Loss of electric power and loss of backup power
- Pump failure
- Damage to sedimentation basins
- Damage to chlorination tanks
- Damage to chemical tanks
- Failure of electrical or mechanical equipment or its anchorage
- Failure of piping systems
- Because WTP/WWTPs represent a complex system that consists of many components, failure of one component can lead to the failure of another or to failure of the whole system, also known as a "cascading effect". Cascading effect refers to damage not only to an infrastructure itself but also to the resulting distributions downline.

Damage states that are caused by seismic forces are defined as listed in Table 25 (FEMA 2013a).

| Damage state | Definition | Status | Functionality |
|---|---|---|---|
| DS0 (none) | No damage | Operational | Normal |
| DS1 (minor) | Malfunction of the plant for a short time (less than three days) due to loss of electric power and backup power (if any), considerable damage to various equipment, light damage to sedimentation basins, light damage to chlorination tanks, and/or light damage to chemical tanks. Loss of water quality may occur. | Operational with minor repair | Close to nominal |
| DS2 (moderate) | Malfunction of the plant for about a week due to loss of electric power and backup power (if any), extensive damage to various equipment, considerable damage to sedimentation basins, considerable damage to chlorination tanks with no loss of contents, and/or considerable damage to chemical tanks. Loss of water quality is imminent. | Operational without considerable repair | Reduced |
| DS3 (extensive) | The pipes connecting the different basins and chemical units would be extensively damaged. This type of damage will likely result in the shutdown of the plant. | Operational after significant repair | No |
| DS4 (complete) | The complete failure of all piping and/or extensive damage to the filter gallery. | Not repairable | No |

Table 25. Earthquake damage states for WTP/WWTPs

### 4.3.4.2 Key metric for consideration

A key consideration for WTP/WWTPs is the time that it takes to restore operations after an earthquake. Figure 74 shows the timeline of restoration for WTP/WWTPs (FEMA 2013a) after an earthquake.

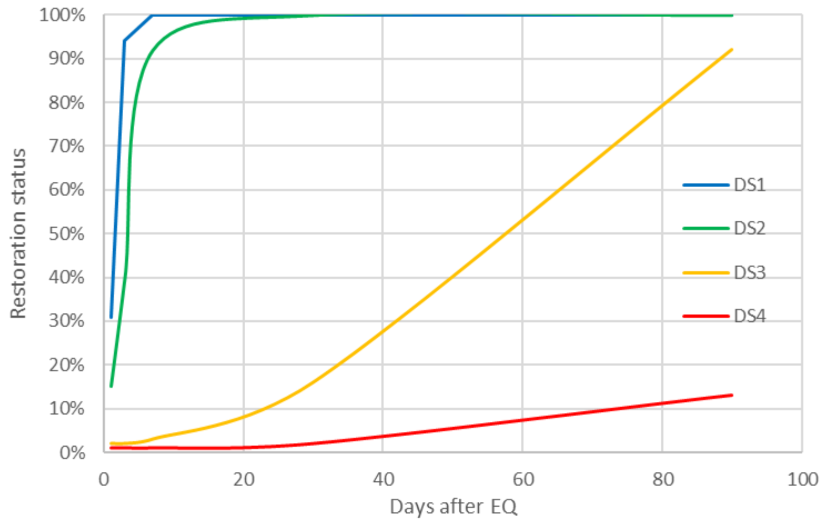



Figure 74. Restoration curve for WTP/WWTPs, earthquake hazard

### 4.3.4.3 Infrastructure improvements

For WTP/WWTPs, several methods can improve performance during an earthquake. Examples include:

- Increase the redundancy of components of WTP/WWTPs through addition of backup components.
- Minimize the possibility of and the effect of cascading failure by introducing emergency shutdown procedures for different failure scenarios.
- Ensure that buildings and mechanical equipment of WTP/WWTPs can tolerate seismic forces.
- Ensure that components of treatment plants cannot be so severely damaged that they then affect the function of an undamaged treatment plant.
- Improve the seismic capacity of other infrastructure facilities, especially the power network, which can adversely affect the function of a WTP/WWTP, a wastewater pumping system or a water pumping system.
- Avoid placing pumping systems in loose sandy soil with a high groundwater table (such as near rivers). Power outages are less likely to severely affect pumping systems or to cause them to fail if such systems are not in these areas.
- To ensure that water and wastewater pipelines cannot fail (leak or break) under a design PGA and the anticipated permanent ground displacement (i.e., liquefaction), make flexible pipes and adequate joint types a top priority for water and wastewater rehabilitation of populated cities in earthquake-prone regions.
- If an earthquake occurs, conduct an intensive post-earthquake inspection to detect all damage in water and wastewater pipelines within the affected zones. Post-earthquake inspections should include various methods to detect all types of earthquake-induced defects in pipelines.
- Ensure that all electrical and mechanical components are designed and are installed per seismic requirements.
- Ensure that a seismic switch is installed to allow safe shutdown and safe restart.
- Perform routine and regular maintenance for the facility and promptly fix any observed problems.

#### 4.3.4.4 Cost-benefit considerations

Panico et al. (201) studied the seismic vulnerability of wastewater treatment plants (WWTPs) and developed fragility curves. Panico et al. (2013) and many studies focus on WWTPs because the consequences of a WWTP failure are more severe than for a WTP. Information about the seismic performance of WTPs is limited, and because WTP and WWTP systems are similar, the vulnerabilities are expected to be similar as well. Panico et al. (2013) concluded that municipal WWTPs tend to be more vulnerable than industrial WWTPs when earthquakes occur. The same damage states can occur at both municipal and industrial WWTPs, but at different risk levels. This difference is a consequence of their varying operational conditions. For an industrial WWTP, the influent flow can be interrupted at any time, but for a municipal WWTP, it can never be interrupted because the community depends on the WWTP for everyday use. Therefore, failure of industrial WWTPs result in a limited release of contaminated water, whereas if a municipal WWTP malfunctions or fails, contaminated water is released into the environment and will continue to be released until the issue is resolved.

Figure 75 presents the fragility functions for both municipal and industrial WWTPs. To address the difference in the failure consequences, the damage states were modified and it was assumed that the seismic capacity (i.e., fragility) would be upgraded from municipal level to industrial level by implementing seismic improvements. For an earthquake with a PGA of 0.35g to 0.4g (the value expected from a $M_w$ 7 earthquake), the damage probability is 70% for unimproved WWTPs and is 40% for improved WWTPs.

Because the performance of WWTPs as a system depends on the performance of individual components, cost-benefit considerations vary greatly, depending on the failure to be mitigated. For example, seismic upgrade of a power unit could preserve the functionality of the system and of the emergency pumps. In this report, the total cost to improve seismic capacity of this component is estimated at approximately 15% of the replacement cost.

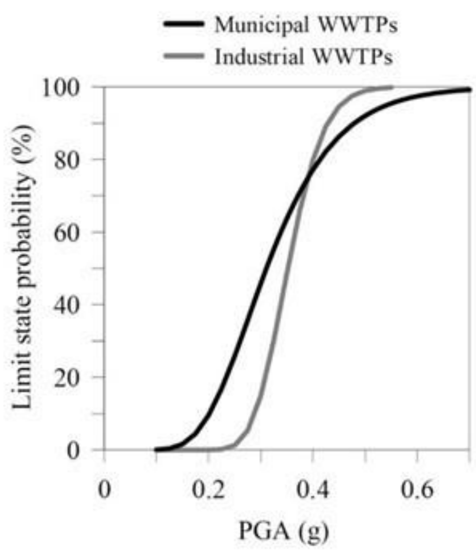



Figure 75. Damage fragility functions for wastewater treatment plants

### *4.3.5 Liquefaction hazard*

#### 4.3.5.1 General

Matsuhashi et al. (2014) conducted an extended study of the damage that the 2011 Great East (Tohoku) Japan Earthquake caused to WWTPs and to sewage systems. One of the study’s main conclusions is that liquefaction-related damage to WWTPs and to pumping stations is much less likely than damage that is caused by seismic events and tsunamis. This conclusion is presented in Figure 76 (Matsuhashi et al. 2014).

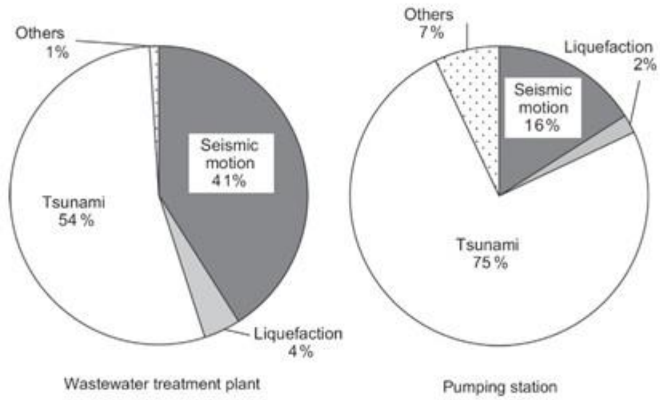



Figure 76. Proportion of damage factors for wastewater treatment plants and pumping stations

However, the effects of natural hazards are completely different for sewage piping systems and manholes. Most failures in sewage piping systems have been caused by liquefaction, as presented in Figure 77(Matsuhashi et al. 2014).

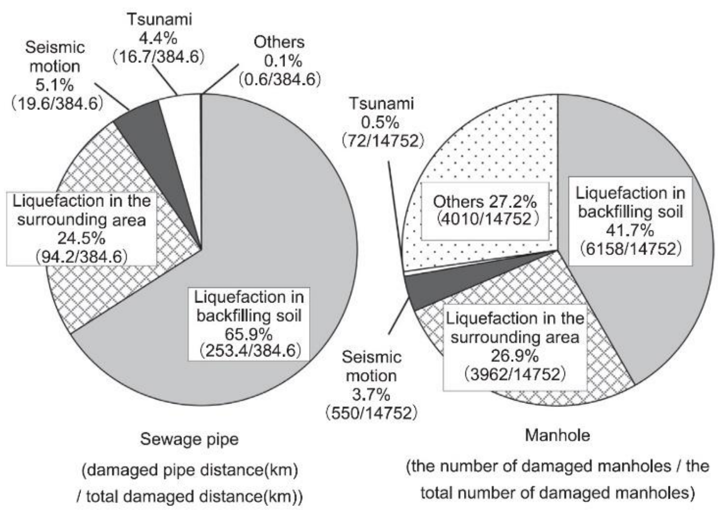



Figure 77. Proportion of damage factors for sewage pipes and manholes

#### 4.3.5.2 Key metric for consideration

A key consideration for WTP/WWTPs is the amount of time that it takes to restore operations. The timeline of restoration for liquefaction-related damage is similar to the timeline for earthquake-related damage. Section 4.3.4.2presents the timeline of restoration for WTP/WWTPs after an earthquake.

#### 4.3.5.3 Infrastructure improvements

The primary recommendation is to build WTPs and WWTPs away from areas that can be susceptible to liquefaction. The following countermeasures against liquefaction-related damage in WTP/WWTPs and in pumping stations should also be considered (Matsuhashi et al. 2014):

- Install equipment at a higher elevation.
- Ensure that the water resistance of the building is adequate, and install water-resistant equipment, doors, and windows.

- Increase the redundancy of critical equipment.

The following countermeasures against liquefaction-related failures of sewage pipes and manholes can be considered (Matsuhashi et al. 2014):

- Compaction of backfilling soil
- Backfilling by crushed stone
- Solidification of backfill

#### 4.3.5.4 Cost-benefit considerations

Because WTPs and WWTPs represent complex systems with many components, which improvement will provide the greatest benefit is plant-dependent. To address this issue, the recommendation is to analyze the plant as a system, to identify the critical components, and to develop possible failure scenarios. For example, based on the analysis, if a critical equipment failure that will cause cascading failure of the system is identified, a solution is to move the plant to an area that is less susceptible to liquefaction; this solution can eliminate this mode of failure completely.

In this report, it is assumed that the damage probability for liquefaction is approximately the same as for earthquake (i.e., 70% and 40%, respectively) because liquefaction damage to WTPs and WWTPs would be induced by an earthquake. The total cost to improve liquefaction resistance, such as through soil improvement, is evaluated to be about 20% of the replacement cost.

### *4.3.6 Wind hazard*

#### 4.3.6.1 General

WTP/WWTPs are short structures and are not vulnerable to damage from wind loading.

#### 4.3.6.2 Key metric for consideration

This topic does not apply.

#### 4.3.6.3 Infrastructure improvements

This topic does not apply.

#### 4.3.6.4 Cost-benefit considerations

This topic does not apply.

### *4.3.7 Flood hazard*

#### 4.3.7.1 General

WTP/WWTPs are susceptible to damage from flooding. Modes of failure can include:

- Loss of electric power and loss of backup power
- Pump failure
- Flooding of sedimentation basins and subsequent contamination
- Damage to chlorination tanks
- Damage to chemical tanks
- Failure of electrical or mechanical equipment

#### 4.3.7.2 Key metric for consideration

A key consideration for WTP/WWTPs is the time that it takes to restore operations after a flood. The timeline of restoration for flood-related damage is similar to the timeline for earthquake-related damage. Section 4.3.4.2 presents the timeline of restoration for WTP/WWTPs after an earthquake. However, depending on the situation, the timeline of restoration could be longer if cleanup of a contaminated site is required.

#### 4.3.7.3 Infrastructure improvements

Two recommended techniques for reducing flood-related damage to essential utility systems and to equipment are to elevate equipment and to implement dry floodproofing (FEMA 2013c).

**Elevate equipment:**

The most effective mitigation method for flood-related damage is to raise all essential equipment to a location that is above the highest anticipated flood elevation, or to the elevation of the 0.2-percent-annual-chance flood that FEMA recommends, whichever is higher. When essential equipment is below grade, elevating typically requires relocating the equipment to higher floors in the building. When elevating equipment is not practical, dry floodproofing might be an option.

**Dry floodproofing:**

Essential equipment can be protected with dry floodproofing methods. Dry floodproofing involves constructing flood barriers or shields around individual pieces of equipment or around areas that contain essential equipment to prevent floodwaters from coming into contact with critical equipment. For dry floodproofing to be effective, the barrier must be high enough to protect equipment from floodwaters, must be strong enough to resist flood forces, and must be sealed well enough to control leakage and infiltration. Dry floodproofing measures must also satisfy applicable codes and standards. Dry floodproofing should meet ASCE 24 criteria (ASCE 2015) in all locations.

#### 4.3.7.4 Cost-benefit considerations

Dry floodproofing and equipment elevation, discussed in Section 4.3.7.3, are among the most cost-effective measures to mitigate flooding failures. Construction of flood barriers or shields around critical equipment or elevation of critical equipment from the ground level is an inexpensive improvement, which can completely prevent failure of the equipment. Building flood barriers around basins represents a much more expensive option that requires more detailed analysis to produce reliable cost-benefit recommendations, which also must be plant-specific. Assuming that damage is proportional to the height of flood inundation, for non-floodproofed/non-elevated equipment, a damage probability of 50% is assumed. By installing water barriers or by elevating equipment, with an estimated 5% improvement cost, the damage probability is then reduced to 20% (i.e., an improvement factor of about 2.5).

## 4.4 Distribution pipes

### *4.4.1 Overview*

Water distribution pipes are typically large pipes (more than 20-in. diameter) or channels (canals) that convey water from its source (reservoirs, lakes, rivers) to a treatment plant.

Transmission water aqueducts are commonly made of concrete, ductile iron, cast iron, or steel, and they can be elevated, at grade, or buried. Elevated or at-grade pipes are typically made of steel (welded or riveted), and they can run in single or multiple lines. Figure 78 shows an example of distribution pipes.

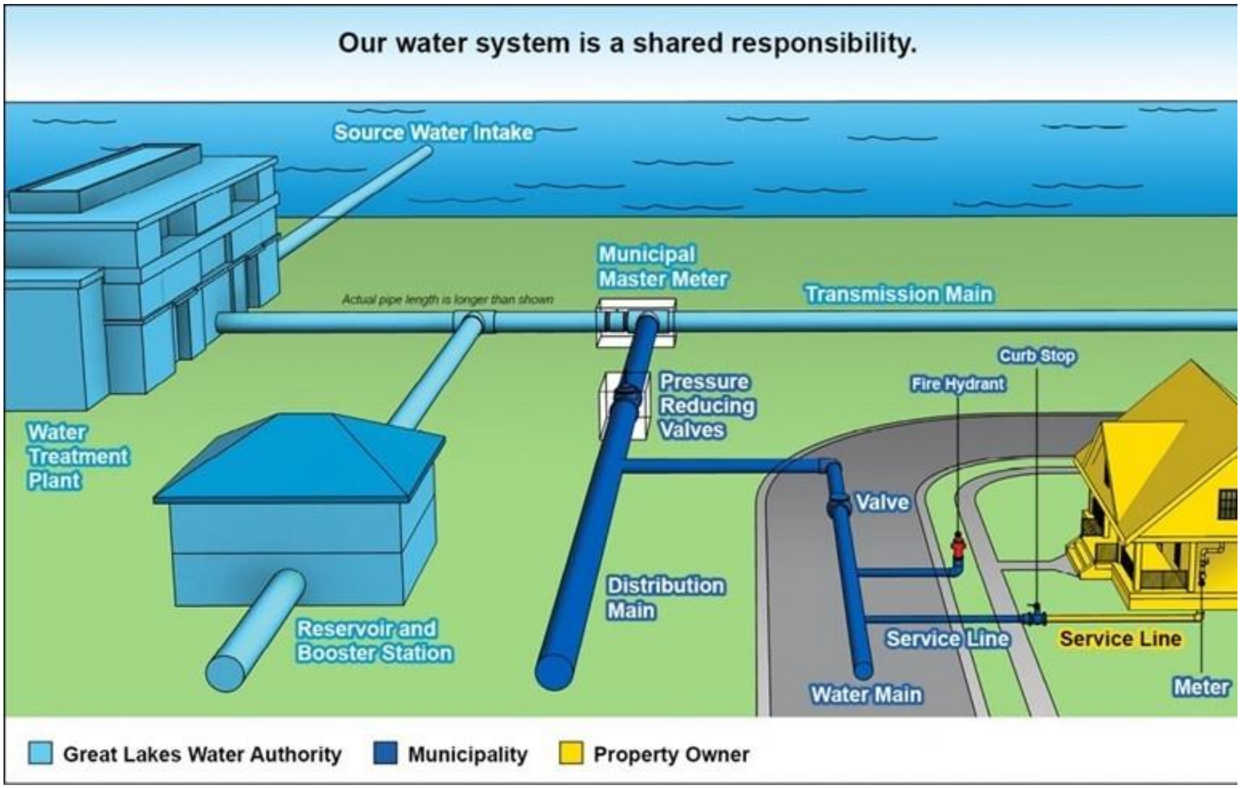



Figure 78. Example of distribution pipes

### *4.4.2 Summary*

The results from a literature review of water pipelines are summarized in Table 26 and are presented in the "*Improvement of infrastructure resilience evaluation matrix for selected natural hazards"* table of the project final report. The following sections of this background report provide more detailed descriptions and background information.

| Hazard | Susceptible | Improvements | Damage index | | Approximate improvement cost as % of cost |
|---|---|---|---|---|---|
| | | | As-is | Improved | |
| Earthquake | Y | Higher threshold in seismic design | 0.7 | 0.4 | 20% |
| Liquefaction | Y | Higher threshold for permanent ground displacement | 0.7 | 0.4 | 55% |
| Wind | N | -- | -- | -- | -- |
| Flood | Y | Higher threshold for large pipe displacement | 0.2 | 0.1 | 2% |

Table 26. Summary of findings for water pipelines

### *4.4.3 Vulnerability to natural hazards*

Water pipelines have been significantly damaged in past natural hazard events. Figure 79 presents examples.

Earthquake (joint failure; Mexico City Earthquake, 1985)

Liquefaction (pipe joint pullout: Kobe, Japan, 1995)

Figure 79. Damage to water pipelines from natural hazards

### *4.4.4 Earthquake hazard*

#### 4.4.4.1 General

Water pipelines are susceptible to damage from both peak ground acceleration/peak ground velocity (PGA/PGV) and permanent ground deformation (PGD). Modes of failure can include:

- Excessive differential movement between the sections
- Failure of joints
- Failure of pipe sections
- Failure at manholes

The fragility curves for buried pipelines are provided in ALA (2001).

For pipelines, two damage states are considered: leaks and breaks. Generally, when a pipe is damaged by ground failure (PGD), the type of damage is likely to be a break. When a pipe is damaged by seismic wave propagation (PGV), the type of damage is likely to be joint pullout or crushing at the bell. In the loss assessment methodology, it is assumed that damage due to seismic waves consists of 80% leaks and 20% breaks, and damage due to ground failure consists of 20% leaks and 80% breaks.

#### 4.4.4.2 Key metric for consideration

The restoration functions for pipelines are expressed in terms of the number of days that are needed to fix the leaks and the breaks (FEMA 2013a); these functions are presented in Table 27. These functions depend on the availability of labor and on the number of breaks and leaks in pipes that are within a certain diameter range. Pipes with a diameter from 60 to 300 in. are assigned the highest priority in restoration. The priority level goes down as the pipe diameter decreases. For example, a pipeline that consists of 60-in. pipes with 10 leaks and 40 breaks can be restored in about 45 days if 20 workers are available to make repairs, as presented in Figure 80. This plot also shows that the restoration process takes much longer if only 10 or 5 workers are available for the repair work.

| Class | Diameter from: [in.] | Diameter to: [in.] | # Fixed breaks per day per worker | # Fixed leaks per day per worker | # Available workers | Priority |
|---|---|---|---|---|---|---|
| a | 60 | 300 | 0.33 | 0.66 | User-specified | 1 (highest) |
| b | 36 | 60 | 0.33 | 0.66 | User-specified | 2 |
| c | 20 | 36 | 0.33 | 0.66 | User-specified | 3 |
| d | 12 | 20 | 0.50 | 1.0 | User-specified | 4 |
| e | 8 | 12 | 0.50 | 1.0 | User-specified | 5 (lowest) |
| u | Unknown or for Default diameter Data Analysis | | 0.50 | 1.0 | User-specified | 6 (lowest) |

Table 27. Restoration functions of water pipelines (FEMA 2013a)

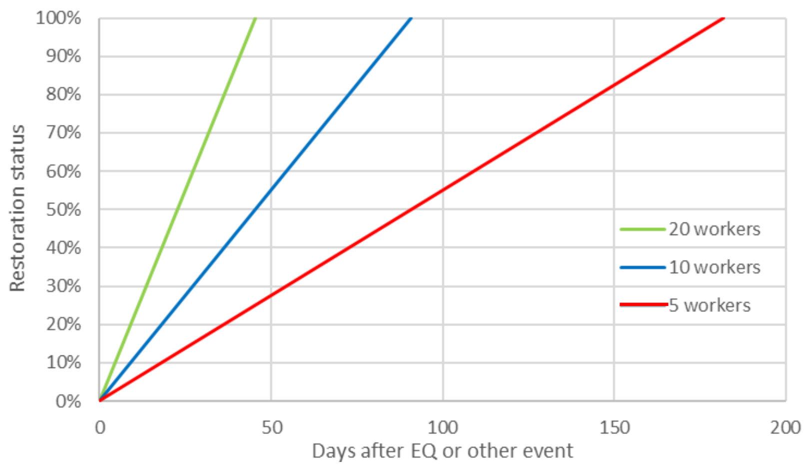



Figure 80. Restoration curve for water pipelines, earthquake hazard

### 4.4.4.3 Infrastructure improvements

For water pipelines, earthquake performance can be improved by several methods. Examples include:

- Replace pipelines that are especially vulnerable to failure from PGD (resulting from liquefaction and landslides).
- Replace pipelines that are made from non-ductile (inflexible) materials, such as concrete and cast-iron pipe, which tend to fail during strong ground motion.
- Create a seismically resilient pipe network.
- Introduce flexible joints, similar to the example in Figure 81 (Haddaway, 2015).

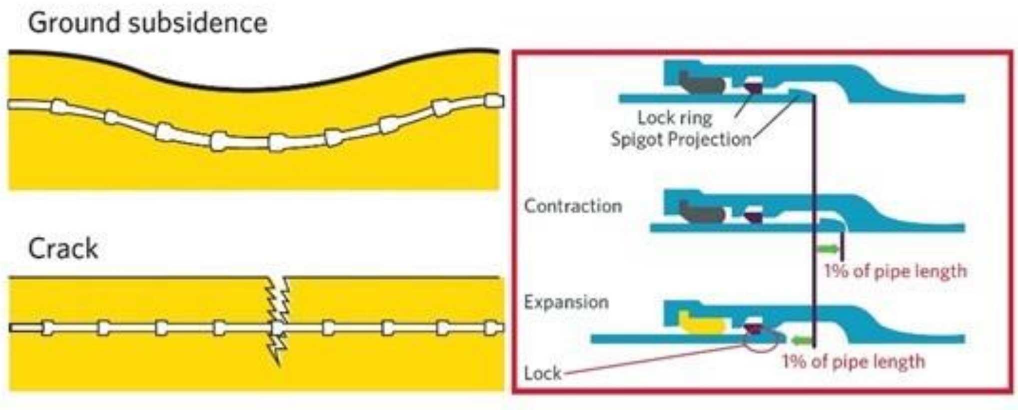



Figure 81. Earthquake-resistant ductile iron pipe (ERDIP) locking joint design (Haddaway 2015, Kubota Corporation)

#### 4.4.4.4 Cost-benefit considerations

Eidinger and Davis (2012) provided the following recommendations and cost-benefit information for improving pipelines to withstand earthquake forces. For seismic mitigation, the long-term strategy is to replace all seismically weak pipes that cross zones that are subject to permanent ground deformation (PGD), such as zones that are subject to liquefaction, landslide, or fault offset. The replacement pipes should be designed to withstand settlement from PGD (such as by using ductile iron pipe with chained joints, butt-fusion-welded or clamped electric-resistance-welded HDPE (High Density Poly-Ethylene) pipe, or heavy-walled butt-welded steel pipe). FEMA (2005) gives specific recommendations for the selection of new pipes in seismic areas. In this report, for existing (unimproved) pipes, a damage probability of 70% is assumed. By implementing the seismic upgrades, with an estimated 20% improvement cost, the damage probability is reduced to 40% (i.e., an improvement factor of about 2).

A pilot project that was undertaken in Los Angeles, California, by Los Angeles Department of Water and Power (LADWP) is a representative example of cost-benefit analysis. The 1994 Northridge Earthquake caused numerous failures of the LADWP network, including: (1) 14 repairs to the raw-water supply conduits, (2) 60+ repairs to treated-water transmission pipes, (3) 1,013 repairs to distribution pipes, and (4) more than 200 service connection repairs (Davis and Castruita 2013). The total cost of repairs was about $US41M. In contrast to those results, the earthquake-resistant ductile iron pipes (ERDIPs) that are used in Japan: (1) have had no damage or leaks after 40 years of use, (2) have survived many large Japanese earthquakes, and (3) have sustained several meters of permanent ground deformation. Replacing the old piping system with ERDIP for the pilot project in Los Angeles (Davis and Castruita, 2013) increases the total cost of the project by about 20%. This increase includes a 13% increase in material costs and about a 7% increase in engineering costs. Although the cost increase is relatively small, the pipeline resiliency increases dramatically, with a very low likelihood of failure.

### *4.4.5 Liquefaction hazard*

#### 4.4.5.1 General

Strong ground motion can cause the development of large pore pressures with an accompanying strength loss that results in liquefaction. Based on the site geometry, the strength loss might induce lateral slope movement, ranging from a few inches to many feet, commonly referred to as a “flow failure.” Liquefaction-induced slope movement often occurs in gently sloping ground toward a free face, such as a creek or a channel. Increases in pore water pressure can impose buoyant force on buried pipelines, which if not properly accounted for, might lead to pipe flotation and possible damage. Liquefaction can also induce pipe flotation, especially for empty pipes, which are commonly used in sewer systems. According to FEMA (2005), flotation has not been a common source of damage for potable water pipelines because they are rarely (if ever) empty. The failure of sewage piping systems is discussed in Section 4.3, related to water treatment plants and wastewater treatment plants.

Liquefaction can cause large lateral permanent deformations of soil or can cause soil settlement in a vertical direction. In these cases, the following failures of buried pipelines can occur:

- Failure of pipe sections: collapse of steel pipes, failure of concrete and asbestos pipes (break-type failures)
- Loss of continuity between pipe sections due to large differential rotation of the sections (leak-type failures)
- Loss of continuity between pipe sections due to large differential lateral movements of the sections (leak-type failures)

#### 4.4.5.2 Key metric for consideration

A key consideration for distribution pipelines is the time that it takes to restore operations after liquefaction damage. The timeline of restoration for liquefaction-related damage is similar to the timeline for earthquake-related damage. Section 4.4.4.2presents the timeline of restoration for water pipelines.

#### 4.4.5.3 Infrastructure improvements

For water pipelines, liquefaction performance can be improved by several methods. Examples include:

- Replace pipelines that are especially vulnerable to failure from permanent ground deformation (resulting from liquefaction and landslides).
- Replace pipelines that are made from non-ductile (inflexible) materials, such as concrete and cast-iron pipe.
- Introduce flexible joints that can accommodate large rotations and lateral movements.
- Improve the properties of backfilling soil for buried pipelines to decrease the pipelines' susceptibility to liquefaction.

#### 4.4.5.4 Cost-benefit considerations

Because of the similarities between earthquakes and earthquake-induced liquefaction, cost-benefit considerations for liquefaction hazards are covered in the earthquake section (Section 4.4.4.4). For liquefaction, a damage probability of 70% before seismic upgrades is assumed, and the damage probability is reduced to 40% (i.e., an improvement factor of about 2) by implementing the seismic improvements, with an estimated 55% improvement cost.

### *4.4.6 Wind hazard*

#### 4.4.6.1 General

Buried water pipelines are not adversely affected by windstorms.

#### 4.4.6.2 Key metric for consideration

This topic does not apply.

#### 4.4.6.3 Infrastructure improvements

This topic does not apply.

#### 4.4.6.4 Cost-benefit considerations

This topic does not apply.

### *4.4.7 Flood hazard*

#### 4.4.7.1 General

During flooding, water or sewer pipelines can experience large displacements due to buoyancy effects only if the pipelines are empty (e.g., during maintenance or repair). For failures of this kind, the information and recommendations in the liquefaction-related section (Section 4.4.5) should be followed.

#### 4.4.7.2 Key metric for consideration

In zones that are subject to flooding, the recommendation is to minimize maintenance or repair time. Because of the similarities in potential failure modes, infrastructure improvements in the liquefaction-related section (Section 4.4.5.3) also apply to flood hazards.

#### 4.4.7.3 Cost-benefit considerations

Because of the similarities to the earthquake-induced liquefaction damage scenario, the cost-benefit considerations in the earthquake section (section 4.4.4.4) also apply to flood hazards. In this report, however, because of the limited occurrence of buoyancy, it is assumed that the flood damage probability is about one-fourth the probability of liquefaction damage. Therefore, a flood damage probability of 0.2 before seismic countermeasures is assumed, and the damage probability is reduced to 0.1 (i.e., an improvement factor of about 3) by implementing the seismic measures, with an estimated 2% cost.

## 4.5 Sewage network emissaries

### *4.5.1 Overview*

Sewage that is discharged from households, factories, commercial facilities, etc., is collected through sewer pipes that are buried underground and is processed in a wastewater treatment plant (WWTP). As shown in Figure 82, a sewage network generally consists of four major components: a catch basin, buried pipe, a pumping station (building and equipment), and a wastewater treatment plant (building and equipment) (FEMA 2013a). Because the network is usually distributed spatially, the natural environment at each component location determines what the most critical natural hazard is.

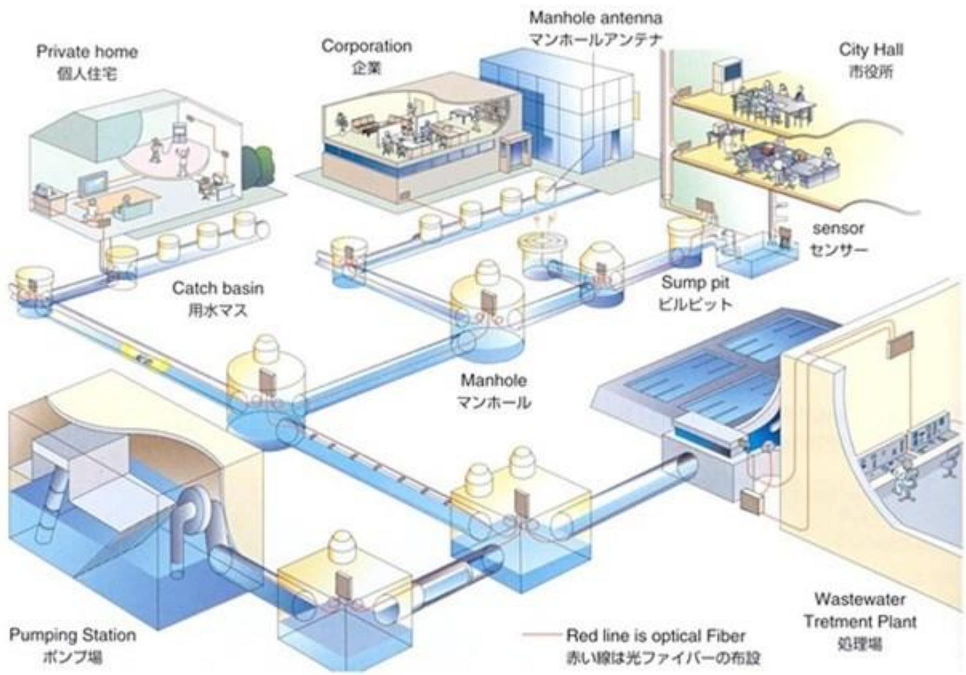



Figure 82. Sewage network

### *4.5.2 Summary*

The results from a literature review of sewage networks are summarized in Table 28 and are presented in the "*Improvement of infrastructure resilience evaluation matrix for selected natural hazards"* table in the final report. In Table 28, the improvement cost is expressed as the ratio of the improvement cost to the component replacement cost, and the resiliency index is estimated as a probability of exceeding severe damage when the hazard threshold intensity occurs. The following subsections of this background report provide more detailed descriptions and background information.

| **Hazard** | **Susceptible** | **Improvements** | **Resiliency index** | | **Approximate improvement cost as % of cost** |
|---|---|---|---|---|---|
| | | | **As-is** | **Improved** | |
| **Earthquake** | Y | Equipment anchorage retrofit | 0.56 | 0.39 | 25% |
| **Liquefaction** | Y | Soil improvement/compaction | 1*1 | 0.3*1 | 55% |
| **Wind** | Y | Connection & envelope retrofit | 0.04 | 0.03 | 15% |
| **Flood** | Y | Elevation & barrier installation | 0.08*2 | 0.01*2 | 40% |

*1: Relative probability is assumed by damage rate (FEMA 2013a).

*2: Probability is assumed by vulnerability curve (FEMA 2013c).

Table 28. Summary of findings for sewage networks

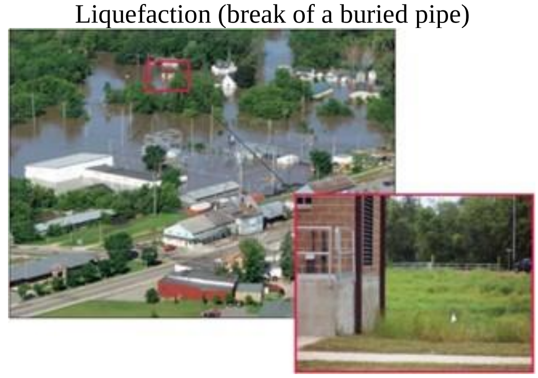
### *4.5.3 Vulnerability to natural hazards*

Equipment and network segments of sewage systems have been subjected to significant damage in past natural hazard events, as shown in Figure 83. Different components of the system are more vulnerable to different types of natural hazards. For example, because of the serious damage to pump stations in past earthquakes (EPA 2018, FEMA 2006, FEMA 2004, and FEMA 2002), the most vulnerable and critical component of a sewage system during seismic shaking is the pump station. If earthquake-induced liquefaction occurs, buried pipes are usually vulnerable to ground displacement and failure, and therefore are assumed to be the most critical component during a liquefaction event (EPA 2018). In a wind hazard, a building is assumed to be a more critical component than pipes or equipment because it has a large area that is loaded by wind (i.e., the roof and walls), especially for a building with more architectural openings, such as a WWTP building (FEMA 2007). In a flood disaster, mechanical/electrical equipment is assumed to be the most critical component, because this kind of equipment is generally weak in water and is automatically switched down to avoid secondary accidents due to water. Thus, this type of equipment at pump stations within a sewage system is considered to be the most critical component in a flood disaster (FEMA 2007).

Earthquake (pump station damage from shaking)

Liquefaction (break of a buried pipe)

Wind (wall wind out-of-plane damage)



Flood (pump station inundation)

Figure 83. Damage to sewage networks from natural hazards

### *4.5.4 Earthquake hazard*

#### 4.5.4.1 General

The level of seismic damage to a pump station (both equipment and buildings) is mainly affected by peak ground acceleration (PGA) from earthquake shaking. The damage states of pump stations after an earthquake can be fundamentally defined as listed in Table 29 (FEMA 2013a).

| Damage state | Definition | Status | Functionality |
|---|---|---|---|
| DS0 (none) | -- | Operational | Normal |
| DS1 (minor) | Malfunction of plant for a short time (less than three days) due to loss of electric power and backup power if any, and/or slight damage to buildings. | Operational | Close to normal |
| DS2 (moderate) | Loss of electric power for about a week, considerable damage to mechanical and electrical equipment, and/or moderate damage to buildings. | Operational with minor repair | Reduced |
| DS3 (extensive) | Buildings are extensively damaged, and/or pumps are badly damaged beyond repair. | Operational after repair | No |
| DS4 (complete) | Building collapsed. | Not repairable | No |

Table 29. Earthquake damage states for a pump station

### 4.5.4.2 Key metric for consideration

The intensity threshold that damages a component (i.e., a pump station) is assumed to be MMI VII–VIII, which is equivalent to a PGA of 0.3g (FEMA 2013a & USGS.gov). A damage state that is higher than DS3 (i.e., more than extensive/severe damage) is used to estimate the resiliency index, such as the damage probability. In addition to the damage probability, a timeline of restoration for a pump station after an earthquake was studied and is shown in Figure 84 (FEMA 2013a).

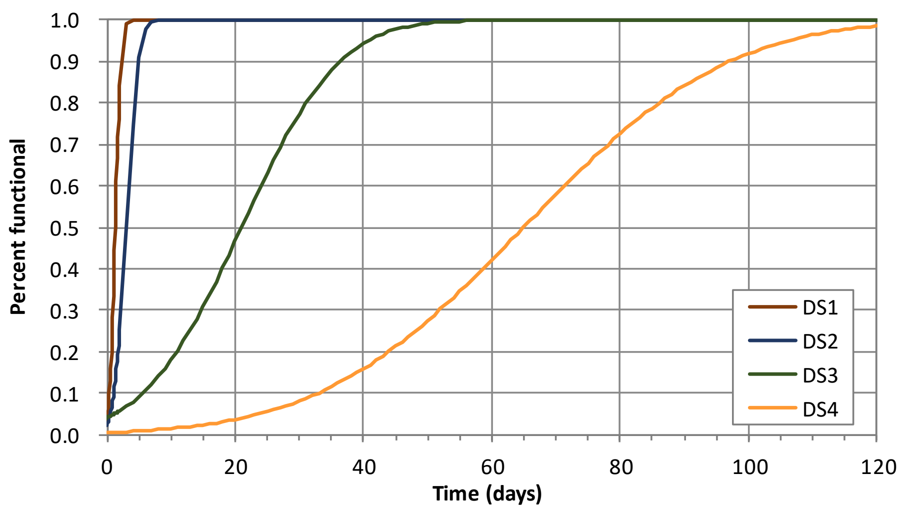



Figure 84. Restoration curve for a pump station following an earthquake

### 4.5.4.3 Infrastructure improvements

The earthquake performance of a sewage network can be improved by several methods, including the methods in the following bullet list. For the most critical component, namely the pump station, anchorage to strengthen the equipment (mentioned in the first bullet) improves the stability and the sustainability of equipment and is an effective engineering improvement for existing pump stations (EPA 2018 & FEMA 2002, 2004, and 2006). Seismic retrofit of a building (e.g., a pump station building) also improves system resiliency. Quality assurance (QA) for equipment or for a facility includes testing (materials testing, anchor capacity testing, weld testing, etc.) and construction inspection. Based on engineering judgment, it is presumed that the cost of standard QA is about 10% of the entire project budget, and a higher level of QA uses approximately 15% of the total budget. For better QA of retrofit work, the higher-level QA approach is considered in this report, and the standard QA is assumed to have been applied to the existing components.

Improvements to increase the earthquake resistance of sewage networks include:

- Properly anchor equipment, including pumps, generators, etc., as shown in Figure 85.
- Ensure that buildings that are part of a sewage network meet seismic code requirements and retrofit these buildings as needed.
- Assess geotechnical components (e.g., catch basins and buried pipes) and strengthen them if any seismic deficiency exists.
- Use seismic protection devices to reduce the load demand on the components or the buildings.
- Ensure that a seismic switch is installed to allow safe stop and safe restart of the machines in operation.
- Perform routine and regular maintenance for all components and fix any observed problems.
- Conduct the higher-level QA protocol (e.g., daily testing and continual and detailed field inspection).

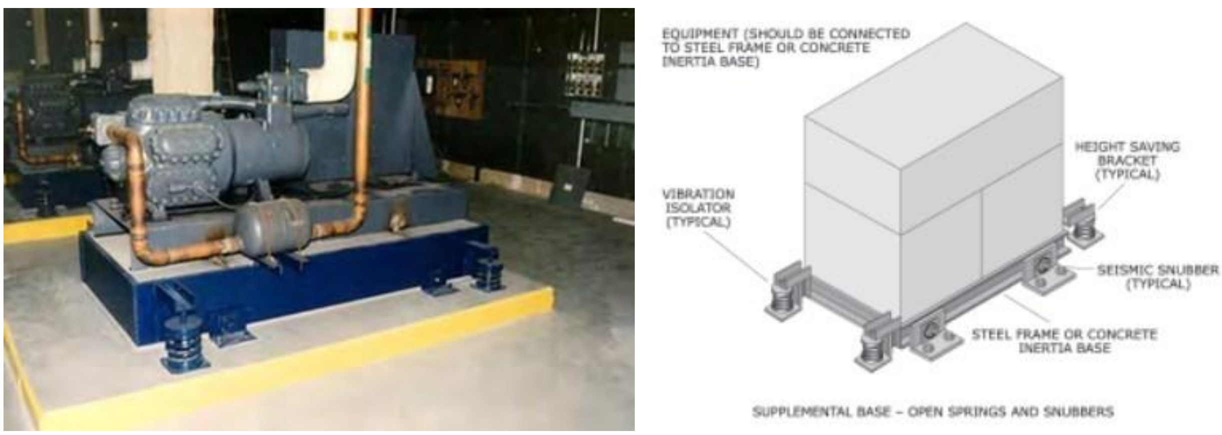
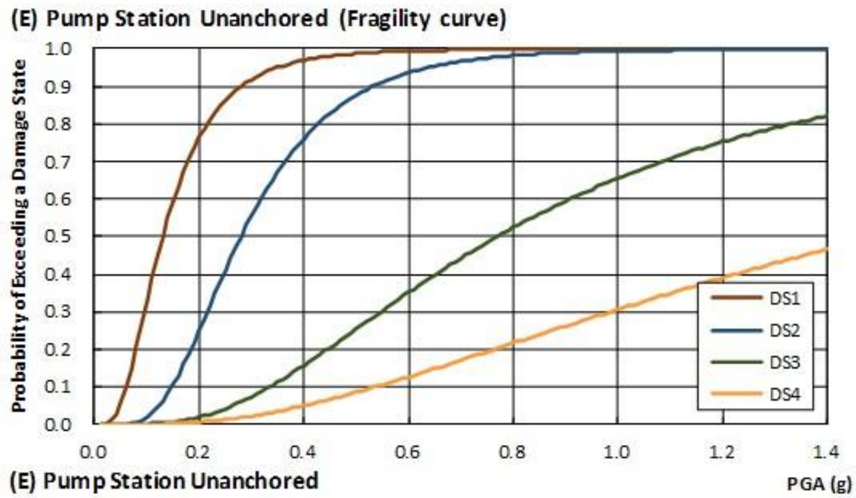



Figure 85. Examples of pump equipment retrofit, anchorage (left) and vibration isolation (right)

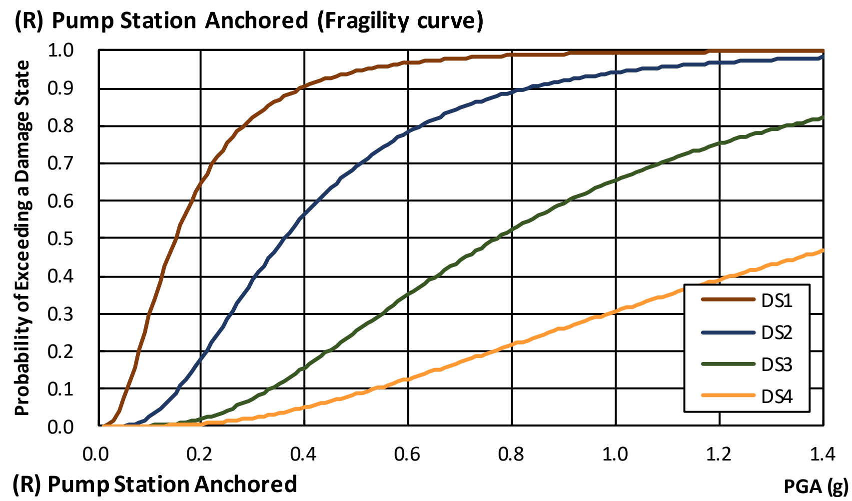
#### 4.5.4.4 Cost-benefit considerations

FEMA (2013a) provides seismic fragility functions for pump stations in use in the United States. Two sets of fragility functions are provided: one for pump stations that are designed conventionally (e.g., unanchored equipment) and one for pump stations with an appropriate seismic design (e.g., appropriate anchorage) that meets the requirements of modern seismic codes. In this report, the fragility parameters of these functions are applied to estimate the damage probability (i.e., resiliency index) of pump stations during seismic motion, and it is assumed that this resiliency index represents similar indices of pump facilities across the world. Figure 86 presents the fragility functions for pump stations before and after improvement. For an earthquake with a PGA of 0.3g (a value that is expected from an earthquake of MMI VII–VIII), the probability of exceeding DS3 is 56% for existing pump stations and is 39% for improved pump stations.

Based on engineering experience and judgment, the cost to improve the seismic capacity (i.e., seismic anchorage) of a pump station is estimated at approximately 24% of the facility replacement cost. The total improvement cost, including the higher-level QA, can then be estimated as 25% of the total cost of replacing the equipment. Because the damage probability is reduced by implementation of the improvement method, the damage level of an improved pump station will likely be reduced during an earthquake. This outcome also means that the restoration period will be shortened, thus providing significant benefits for the sewage network and for community life.




Unanchored subcomponents




Anchored and seismic subcomponents

Figure 86. Seismic fragility functions for pump stations of sewage networks

### *4.5.5 Liquefaction hazard*

#### 4.5.5.1 General

Pipes that are buried below ground can be damaged by large permanent ground deformation (PGD) such as horizontally forced displacement and lateral spreading of soil due to earthquake-induced liquefaction (FEMA 2013a). Pipe networks that are installed in liquefiable and weak soils in high-seismicity areas are especially vulnerable to liquefaction damage.

#### 4.5.5.2 Key metric for consideration

When the intensity threshold that damages a buried component (i.e., a sewage pipe) is assumed to be a PGD of 12 in. (FEMA 2013a, Ferritto 1997, Urayasu 2012, & Nakashima et al. 2015), a damage state of higher than DS3 (i.e., more than extensive/severe damage) is adopted to represent the resiliency index, such as damage probability. Liquefaction can damage sewage networks and result in loss of sewage network capacity, and because the failures normally occur underground, the cost of geotechnical repair and restoration of the pipe network is high.

### 4.5.5.3 Infrastructure improvements

For sewage networks, liquefaction resistance can be improved by several measures, including the methods in the following bullet list. For the most critical component in a liquefaction occurrence (i.e., buried pipes), the proposed soil improvement method in the first bullet can greatly increase the liquefaction capacity and is an effective engineering improvement for existing buried pipes (EPA 2018 & Andrus and Chung 1995). This method aims to strengthen the weak soil by permeation grouting or by jet grouting. As stated in Section 4.5.4.4, it is assumed that the standard-QA cost is about 10% of the project budget, and the higher-level QA cost is approximately 15% of the total budget for a sewage network. Therefore, for better QA of retrofit work, the higher-level QA approach is considered in this report, as shown in the last bullet, and the standard QA is assumed to have been applied to the existing components.

Improvements to increase the liquefaction resistance of sewage networks include:

- Apply soil improvements and densification by several measures (e.g., cement mixing and soil compaction); see Figure 87
- Use a deep foundation, such as pile, wall pile, or piled raft.
- Replace the existing pipes with ductile and flexible-joint pipes.
- Move pipe networks into non-liquefiable areas.
- Conduct the higher-level QA protocol (e.g., daily testing and continual and detailed field inspection).

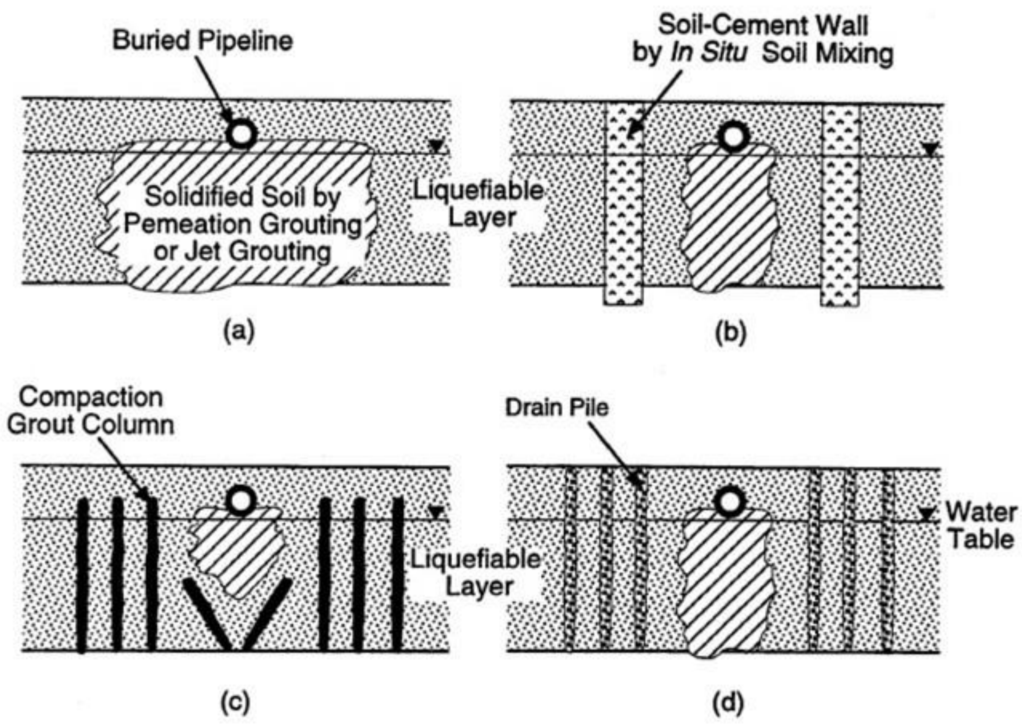



Figure 87. Examples of soil improvement

### 4.5.5.4 Cost-benefit considerations

FEMA (2013a) proposes a seismic damage rate for buried pipes that are laid in the United States. Two sets of damage rates are provided: one for brittle pipes and one for ductile pipes that have higher seismic capacity. These rates are based on a certain level of earthquake. Because in this report the resiliency index (i.e., damage probability) is defined as an occurrence rate of damage during a hazard event, it is assumed that these pipe damage rates can represent the resiliency index before and after implementation of improvement measures against liquefaction. Therefore, the damage rates in Table 30 (FEMA 2013a) are applied as the damage probability (i.e., resiliency index) for buried pipes from seismic liquefaction. For damage that is caused by liquefaction displacement, the damage rates (i.e., probability of damage occurrence, in this report) are 1.0 and 0.3 for the pipe-buried soil before and after improvement, respectively.

Although the cost to improve resistance to earthquake liquefaction (i.e. soil improvement and compaction) greatly depends on local constructability, site complexity, and environmental factors, in this report, it is estimated at approximately 50% of the pipe replacement cost. The total improvement cost, including the higher-level QA, can then be estimated as 55% of the total cost of buried pipe construction. The damage rate is significantly reduced by the improvement measures, and the damage level of pipes will likely be reduced if an earthquake occurs. Presumably, the recovery process for the sewage system will also be faster with these improvements.

| | **PGV algorithm** | | **PGD algorithm** | |
|---|---|---|---|---|
| | R.R. = 0.0001 x PGV$^{(2.25)}$ | | R.R. = Prob[liq] x PGD$^{(0.56)}$ | |
| Pipe type | Multiplier | Example of pipe | Multiplier | Example of pipe |
| Brittle sewers/interceptors | 1 | Clay, concrete | 1 | Clay, concrete |
| Ductile sewers/interceptors | 0.3 | Plastic | 0.3 | Plastic |

Table 30. Seismic damage rate of different pipes (FEMA 2013a)

### *4.5.6 Wind hazard*

#### 4.5.6.1 General

During a wind hazard event, the largest area that is loaded by wind, such as the roof and walls of a WWTP building, is most susceptible to critical damage. This critical damage is caused mainly by peak gust wind speed (PGWS) during a hurricane or a typhoon (FEMA 2013b). Wind damage can be due to direct wind and cross wind, also known as “vortex shedding.”

#### 4.5.6.2 Key metric for consideration

Based on a review of design codes and relevant literature (FEMA 2013b, NSSL, & Beaufort Number), the intensity threshold that damages a critical component is assumed to be a PGWS of 90 mph. Also, the extensive/severe damage state is adopted to scale the resiliency index (i.e., damage probability), which is estimated by the fragility curves in Figure 90 Such a strong wind can result in the loss of functionality at the WWTP, and therefore the facility is unlikely to remain operational. Relative to other types of sewage network damage, the effort required to repair and to restore wind-damaged stations is not so large.

#### 4.5.6.3 Infrastructure improvements

For sewage systems, the wind load capacity can be reinforced by several methods, including the measures in the following bullet list. For the most critical component in a wind occurrence, the WWTP building, roof-wall connection retrofit and building envelope strengthening, detailed in the first bullet, improve the wind load capacity and are effective engineering improvements for existing WWTP buildings (FEMA 2007 & FEMA 2010). These retrofit solutions work to strengthen the roof-wall connection and to renew the coverings on windows, doors, and walls to resist the current design wind load. Again, based on engineering judgment, it is similarly assumed that the standard-QA cost is about 10% of the project budget, and the higher-level QA cost is approximately 15% of the total budget. Therefore, for better QA of retrofit work, the higher-level QA approach is considered in this report, as shown in the last bullet, and the standard QA is assumed to have been applied to the construction of existing components.

Improvements to increase the wind load resistance of sewage networks include:

- Retrofit roof-wall connections and replace building envelopes; see Figure 88 and Figure 89.
- Strengthen the components of the building type for a higher design wind speed and load.
- Ensure that all equipment (e.g., pumps, generators, and pipes) is properly anchored by using positive mechanical attachments that are wind-rated.
- Confirm the wind resistance of the buildings that constitute the sewage network and retrofit buildings if necessary.
- Conduct the higher-level QA protocol (e.g., daily testing and continual and detailed field inspection).

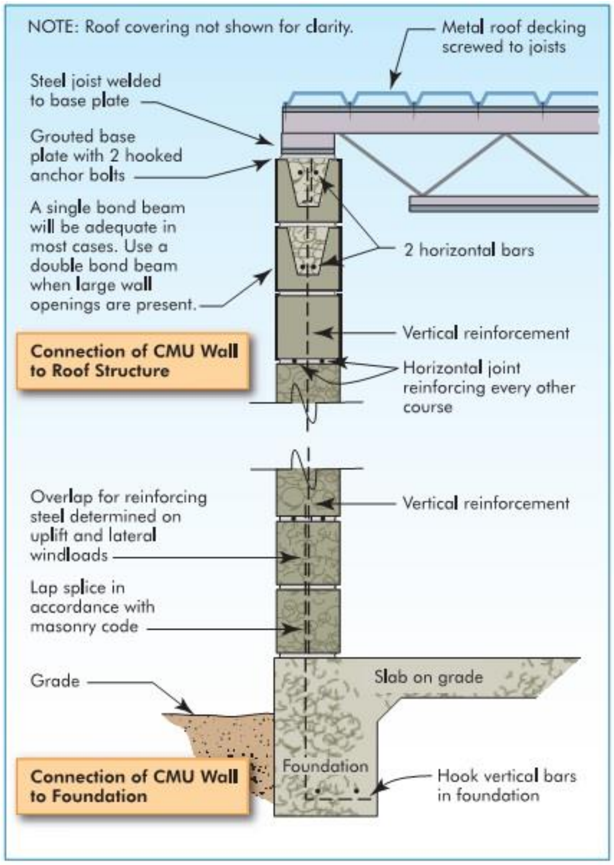



Figure 88. Example of appropriate roof-wall connection

Accordion shutter for windows

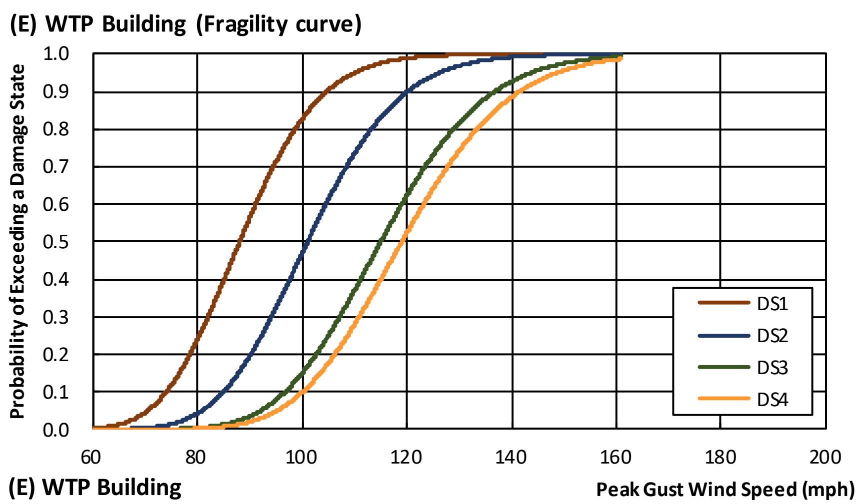
Robust door/shutter

Figure 89. Example of building envelope improvement

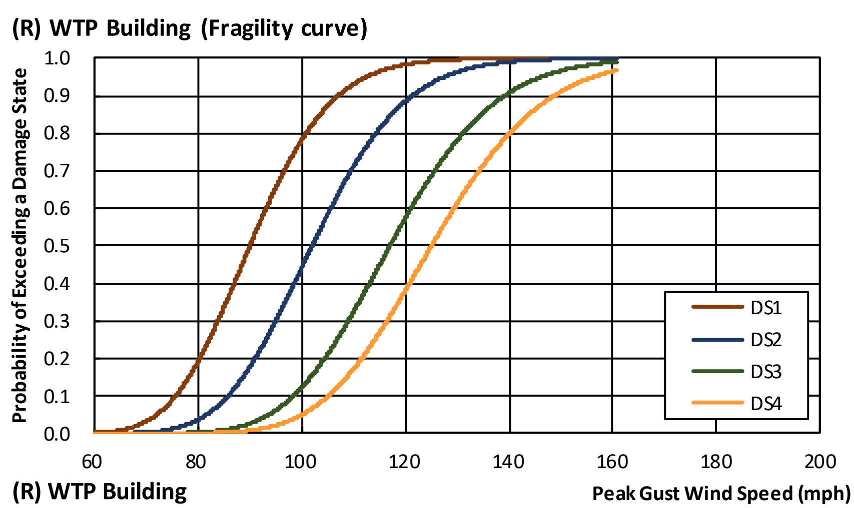
### 4.5.6.4 Cost-benefit considerations

FEMA (2013b) provides wind fragility functions for buildings that are similar to WWTP buildings in the United States. Two sets of fragility functions are provided: one for buildings with low-capacity roof-wall connections and one for buildings whose roof-wall connection is stronger and more resilient. In this report, the difference between these fragility functions is considered to reflect the improvement from implementation of wind strengthening methods. The fragility parameters of these functions are then adopted to estimate the damage probability (i.e., resiliency index) for WWTP buildings from wind hazards.

Figure 90 illustrates the fragility functions for WWTP buildings before and after improvement, respectively. For a strong wind with a PGWS of 90 mph, the probability of exceeding DS3 is 3.5% for existing WWTP buildings and is 2.6% for improved WWTP buildings. In this report, the cost of improvement to sustain a wind hazard (i.e., roof-wall connection retrofit and building envelope replacement) is estimated at approximately 12% of the WWTP building replacement cost (FEMA 2010). The total improvement cost, including the higher-level QA, can then be estimated as 15% of the total cost of the WWTP building construction. The damage probability and the likely damage level are reduced by the improvement measures; therefore, the recovery process for a WWTP building from a strong wind event will likely be faster.




Existing wastewater treatment plant (WWTP)




Retrofitted WWTP

Figure 90. Wind fragility functions for WWTPs of sewage networks

### 4.5.7 Flood hazard

#### 4.5.7.1 General

The extent of flood damage to a pump station is greatly affected by the flood inundation depth (FID) that severe flooding causes. Flooding typically damages electrical and mechanical equipment such as pumps and generators in the pump station of a sewage network. Consequently, this equipment failure causes a bottleneck, which leads to cascading loss of pump station operation (FEMA 2013c).

#### 4.5.7.2 Key metric for consideration

The intensity threshold that damages the pump station (i.e., mechanical/electrical equipment in a pump station) is assumed to be an FID of 3.3 ft. (FEMA 2013c, U.S. Army Corps 2017, Huizinga et al. 2014, & MLIT 2014). The damage state for a flood hazard is assumed to be either damaged (non-operable) or undamaged (operable), and the damage state is adopted to represent the resiliency index, estimated by damage probability. Primarily, electrical and mechanical equipment is vulnerable to flood disasters, and the malfunction of these types of equipment directly causes disruptions or outages in sewage networks.

#### 4.5.7.3 Infrastructure improvements

For sewage networks, the capability to resist flood-related damage can be improved by several methods, including the measures in the following bullet list. For the most critical component, the pump station, equipment elevation and installation of a watertight barrier can improve flood resistance and are recommended engineering improvements for existing pump stations (FEMA 2013d, FEMA 1999, & FEMA 2007). These improvements include moving the pumps and other equipment to a higher position within the building or constructing a pedestal and installing an enclosure wall that serves as a watertight barrier. As proposed in other hazard improvement sections, the standard-QA cost is considered to be about 10% of the project budget, and the higher-level QA cost is approximately 15% of the total budget. Therefore, for better QA of retrofit work, the higher-level QA approach is considered in this report, as shown in the last bullet. The standard QA is assumed to have been applied to the existing components.

Improvements to increase the flood resistance of sewage networks include:

- Elevate the components (e.g., equipment) and install a watertight barrier; Figure 91.
- Install flood-monitoring sensors to notify operators when flooding occurs and as it reaches certain water levels for critical components.
- Relocate critical components and equipment to a flood-safe location (e.g., a higher position).
- Duplicate the path of critical components and equipment to provide disaster resiliency.
- Estimate the potential inundation depth at each building location and prepare suitable countermeasures.
- Conduct the higher-level QA protocol (e.g., daily testing and continual and detailed field inspection).

Equipment elevation

Watertight barrier

Figure 91. Example of equipment elevation and a watertight barrier

### 4.5.7.4 Cost-benefit considerations

FEMA (2013c) proposes a damage percentage for a pump station according to FID, as shown in Figure 92. The percentage is based on a certain depth of flood inundation; 6 ft. is the deepest expected inundation for this type of component. In this report, it is assumed that the conditional pump damage rates from a flood event represent the damage probability for this component (i.e., conditional relative probability), and the differences between existing and improved components are evaluated by different inundation depths. The damage rates in Figure 92 (FEMA 2013c) are then applied as the damage probability (i.e., resiliency index) for a pump station from a flood disaster.

Based on the proposed improvement methods in Section 4.5.7.3, the possible inundation depth can be greatly reduced. It is assumed that implementation of the engineering improvements will lower the FID from 3.3 ft. for existing components to 0.5 ft. for improved components. For an FID of 3.3 ft., the damage rate (i.e., probability of damage occurring) is 8%, as Figure 92 shows. After implementation of improvements, the FID is 0.5 ft., and the damage rate is 0.5% for a pump station within a sewage network.

The cost to improve flood hazard resistance (i.e., equipment elevation and watertight barrier installation) is estimated as 40% of the cost to replace the pump station itself entirely (FEMA 1999). The total improvement cost, including the higher-level QA, can then be estimated as 40% of the total cost to construct a pump station. The resiliency index can be greatly improved by the strengthening methods, so sewage network malfunctions due to pump station damage can be rectified more quickly. Such expedited recovery helps avoid considerable economic loss and improves public benefits.

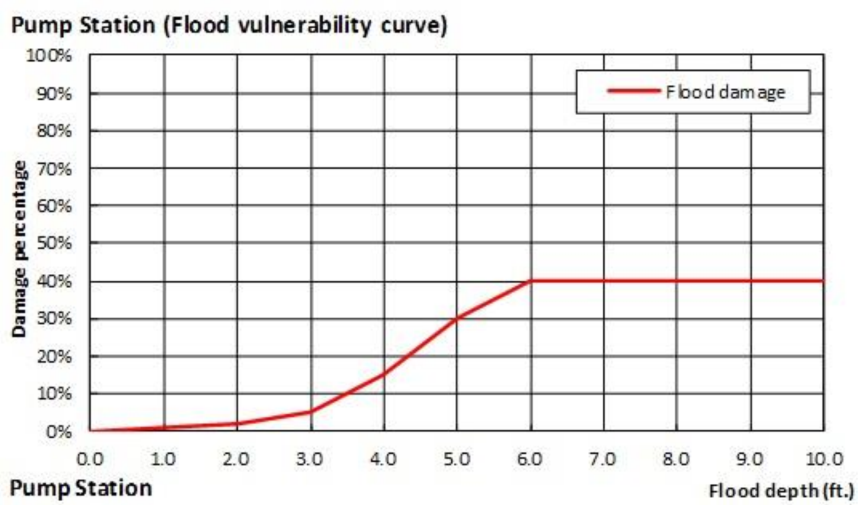



Figure 92. Flood vulnerability function

## 4.6 Water conveyance systems (canals)

### *4.6.1 Overview*

Canals, artificial channels, and waterways are used to convey water from its source, such as a reservoir, to customers. Figure 93 shows an example of a water conveyance system. Open canals usually follow the slope of the terrain and have trapezoidal or rectangular cross sections. Canals are typically lined with concrete to reduce water loss and seepage. Expansion joints are used to allow movement from thermal expansion and shrinkage. Culverts are buried pipes that provide transitions for water flow underneath obstacles such as roadways. Culvert design, vulnerability, and improvements are similar to pipeline networks; see Section 4.4 of this report for more information.

Figure 93. Water canal (Arizona and California, USA)

### *4.6.2 Summary*

The results from a literature review of water conveyance are summarized in Table 31 and are presented in the "*Improvement of infrastructure resilience evaluation matrix for selected natural hazards"* table in the final report. The following subsections of this background report provide more detailed descriptions and background information.

| Hazard | Susceptible | Improvements | Damage index | | Approximate improvement cost as % of cost |
|---|---|---|---|---|---|
| | | | As-is | Improved | |
| **Earthquake** | Y | Reinforced concrete liners | 0.2 | 0.05 | 20% |
| **Liquefaction** | Y | Soil improvement | 0.2 | 0.01 | 3% |
| **Wind** | See note‡ | -- | -- | -- | -- |
| **Flood** | Y | Add floodgate and dry canal | 0.1 | 0.02 | 15% |

Table 31. Summary of findings for water conveyance

‡ The canal itself is not vulnerable to wind damage. However gates could be and as such brief discussion is provided in this section.

### *4.6.3 Vulnerability to natural hazards*

Water conveyance systems have been significantly damaged in past natural hazard events. Figure 94 shows examples of damage to water canals.

Earthquake (Mexicali, Mexico, 2010)

Liquefaction (Mexicali, Mexico, 2010)

Wind (Hurricane Irene, USA, 2011)

Flood (Chennai City, India, 2016)

Figure 94. Damage to water canals from natural hazards

### *4.6.4 Earthquake hazard*

#### 4.6.4.1 General

Canals are susceptible to damage from peak ground velocity (PGV) and permanent ground displacement (PGD) from wave propagation and permanent deformation. Damage states are defined as listed in Table 32 (ALA 2001a & ALA 2001b).

| **Damage state** | **Definition** |
|---|---|
| DS0 (none) | The canal has the same hydraulic performance after the earthquake. |
| DS1 (minor) | 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 (25 to 130 mm) 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. Overall, the canal can be operated at up to 90% of capacity without having to be shut down for make repairs. |

| Damage state | Definition |
|---|---|
| DS2 (moderate) | 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 |
| DS3 (major) | The canal is damaged to such an extent that immediate shutdown is required. The damage may be due to PGDs (150 mm or more) 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 |

Table 32. Earthquake damage states for water canals

### 4.6.4.2 Key metric for consideration

The hydraulic performance of a canal is the key metric to consider. The repair rate per kilometer (RR/km) depends on the damage state. Table 33 presents the repair rates (ALA 2001a & ALA 2001b) for DS1 for water canal systems.

| **PGV, m/sec** | **RR/km** | |
|---|---|---|
| | **Unlined** | **Concrete lined** |
| <0.15 | 0 | 0 |
| 0.15–0.4 | 0.01 | 0.0025 |
| 0.5–0.9 | 0.1 | 0.025 |

Table 33. Repair rates for minor damage to canal systems, earthquake hazard

### 4.6.4.3 Infrastructure improvements

For water canals, earthquake performance can be improved by several methods. Examples include:

- Account for larger earthquake velocities and displacements.
- Use reinforced concrete lining.

### 4.6.4.4 Cost-benefit considerations

ALA (2001a and 2001b) provides damage estimates for water canals based on PGV and PGD. Two sets of data are provided: one for canals without liners and one for canals with reinforced concrete liners. Table 34 presents the vulnerability functions for both cases. Note that for an earthquake with a PGV of 0.5 m/sec. (a value expected from an $M_w$ 6 to $M_w$ 7 earthquake), the canal with a reinforced concrete lining is likely to have no damage. However, canals that have no lining or that have an unreinforced concrete lining can lose up to 20% of their capacity. In other words, by using seismic components (albeit at a higher cost, estimated at approximately 20% in this report), the probability of full water transmission is increased from approximately 80% to 100%, and the repair rate is decreased by a factor of 4.

| | **Wave propagation (PGV), m/sec** | | **Permanent deformation (PGD), m** | | |
|---|---|---|---|---|---|
| Value | <0.5 | ≥0.5 | <0.025 | 0.025–0.15 | 0.15 |
| Unreinforced liner or unlined | DS0 | DS1 | DS0/DS1 | DS2 | DS3 |
| Reinforced liners | DS0 | DS0 | -- | -- | -- |

Table 34. Vulnerability of water canals to earthquake effects

### *4.6.5 Liquefaction hazard*

#### 4.6.5.1 General

Water canals are constructed over long stretches, and portions of them can be on liquefiable soil. Damage can include loss of bearing strength, differential settlement, and soil spreading.

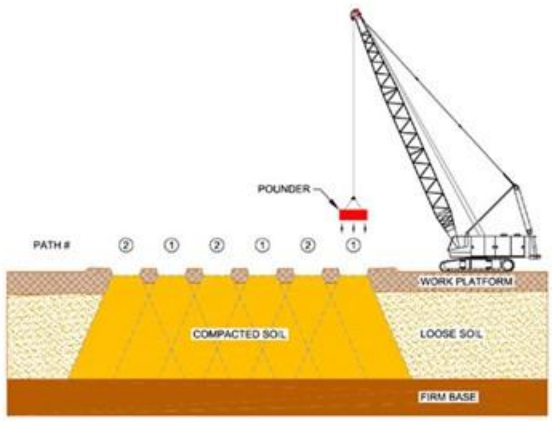
#### 4.6.5.2 Key metric for consideration

The key metrics to consider are the structural integrity of the canal and the canal's ability to transport water to full capacity.

#### 4.6.5.3 Infrastructure improvements

Localized ground failures are smaller-scale displacements that often can be mitigated by treating the site with soil improvement methods (CGS 2008). Suitable mitigation alternatives might include one or more of the following (see Figure 95):

- Excavate and remove or re-compact potentially liquefiable soils.
- Perform soil densification or other types of in-situ ground modifications.
- Reinforce shallow foundations and improve the structural design to withstand the predicted vertical and lateral ground displacements.
- Use ductile channel liners to allow differential movement.




Soil densification

Geomembrane liners

Figure 95. Examples of liquefaction mitigation

#### 4.6.5.4 Cost-benefit considerations

It is anticipated that liquefaction will mainly be localized, and most of the canal will be constructed away from liquefiable zones. Thus, the cost of localized mitigation is estimated at 3% of the cost of replacement. However, because localized failure can jeopardize the operation of the entire canal, the damage is estimated to be 20% for the unimproved configuration and 1% for the improved configuration.

### *4.6.6 Wind hazard*

#### 4.6.6.1 General

Canals themselves are not vulnerable to damage from wind loading, and they are not large enough to be subject to water-wave generation from windstorms. If the canals have gates or locks, these components could experience damage during windstorms. Figure 96 shows two examples of canal gates.

Figure 96. Canal gates

#### 4.6.6.2 Key metric for consideration

The key metric is whether gates (and locks) can remain functional so that the canal can continue to operate.

#### 4.6.6.3 Infrastructure improvements

Examples of wind mitigation include:

- Design gates for the expected (larger than 100-mph) wind at the site.
- Ensure that the gates are properly secured.

#### 4.6.6.4 Cost-benefit considerations

Because the canal itself is not susceptible to wind damage, the costs and benefits of improvements are marginal.

### *4.6.7 Flood hazard*

#### 4.6.7.1 General

Table 35 (FEMA 2013c) presents the vulnerability of canals to flooding, adopted from data from wells to open channels.

| Overall vulnerability | Inundation | Scour | Hydraulic pressure & debris | Financial loss | Loss of operation |
|---|---|---|---|---|---|
| High | High | None | High | High | High |

Table 35. Flood vulnerability for canals (adopted from free-flowing impounds)

#### 4.6.7.2 Key metric for consideration

The status of damage from flooding (adopted from data from wells to open channels) (FEMA 2013c) for canals is presented in Figure 97.

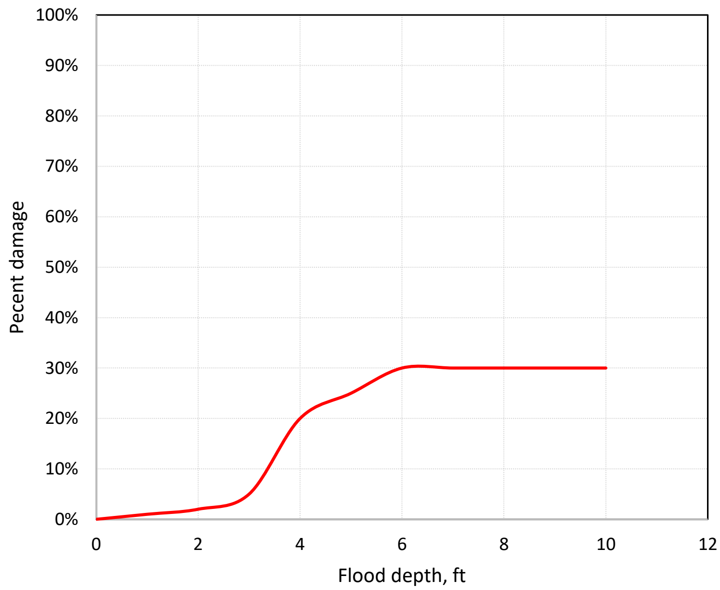



Figure 97. Percentage of damage as a function of inundation depth

#### 4.6.7.3 Infrastructure improvements

The following improvements can be made to reduce damage to canals from flooding:

- Provide reinforced concrete liners.
- Provide sufficient freeboard to accommodate added water.
- Perform regular maintenance, keep the canal clean, and remove all debris and obstructions.
- Provide floodgates, flume gates, and weirs to allow water discharge during storm surges or during heavy water flow; see Figure 98.
- Next to canals, provide emergency water storage or dry canals that can be used in case of flooding.

Canal weirs

Vertical floodgate

Figure 98. Example of flood mitigation

#### 4.6.7.4 Cost-benefit considerations

The additional cost of providing a vertical floodgate and adjacent dry channel storage is estimated at 15%. It is estimated that such improvements will reduce flood damage from 10% for existing conveyance systems to 2% for improved systems

### 4.7 Drainage systems

#### *4.7.1 Overview*

A drainage system is a water infrastructure network for discharging water that has accumulated in buildings, facilities, soil, etc., to the outside. The discharged water is generally called "drainage." Typically, the system is spread widely, as illustrated in Figure 99, and it generally consists of three fundamental below-ground components: the catch basin, the drain channel, and the drainpipe. For this kind of expansive infrastructure system, a multi-hazard analysis is necessary; serious hazards are governed by the natural environment at each infrastructure component location, and each system component has a different vulnerability and robustness for each hazard.


Figure 99. Drainage system

#### *4.7.2 Summary*

The results from a literature review of drainage systems is summarized in Table 36 and are presented in the "*Improvement of infrastructure resilience evaluation matrix for selected natural hazards"* table in the final report. In Table 36 the improvement cost is simply expressed as the ratio of the improvement cost to the component replacement cost, and the resiliency index is estimated as the probability of exceeding severe damage when the hazard threshold intensity occurs. The following subsections of this background report provide more detailed descriptions and background information.

| **Hazard** | **Susceptible** | **Improvements** | **Resiliency index** | | **Approximate improvement cost as % of cost** |
|---|---|---|---|---|---|
| | | | **As-is** | **Improved** | |
| **Earthquake** | Y | Pipe & joint replacement | 1*1 | 0.3*1 | 105% |
| **Liquefaction** | Y | Soil improvement/compaction | 1*1 | 0.3*1 | 55% |
| **Wind** | N | -- | -- | -- | -- |
| **Flood** | N | -- | -- | -- | -- |

*1: Relative probability is assumed by damage rate (FEMA 2013a).

Table 36. Summary of findings for drainage systems

### *4.7.3 Vulnerability to natural hazards*

The link components of drainage systems have been significantly damaged in past earthquakes and from liquefaction, as exemplified in Figure 100. For an earthquake, the most critical component during seismic shaking is assumed to be the buried drainpipe, because joint pullout and bell crushing of drainpipes have been reported as serious damage during past seismic events (EPA 2018 & FEMA/NIBS 2005). In earthquake-induced liquefaction, buried drainpipes are also vulnerable to ground displacement and failure that is caused by unstable movement of soil, and therefore can be assumed to be the most critical component in a liquefaction event (EPA 2018).

Earthquake (pipe joint pullout)

Liquefaction (pipe break)

Figure 100. Damage to drainage systems from natural hazards

### *4.7.4 Earthquake hazard*

#### 4.7.4.1 General

The level of seismic damage to drainpipes is greatly affected by peak ground acceleration (PGA) and peak ground velocity (PGV) from earthquake shaking. The damage states of drainpipes can be defined as follows (FEMA 2013a):

> *For pipelines, two damage states are considered. These are leaks and breaks. Generally, when a pipe is damaged due to ground failure, the type of damage is likely to be a break, while when a pipe is damaged due to seismic wave propagation, the type of damage is likely to be joint pull-out or crushing at the bell.*

#### 4.7.4.2 Key metric for consideration

The intensity threshold that damages the component (i.e., drainpipe) is assumed to be MMI VII–VIII, which is equivalent to a PGA of 0.3g (FEMA 2013a & USGS.gov). A damage state that represents more than extensive/severe damage is used to estimate the resiliency index (i.e., damage probability). In addition to the damage probability, a restoration function for a drainpipe after an earthquake was researched and is presented in Table 37 (FEMA 2013a). Based on the findings, the recovery time frame for drainpipe damage greatly depends on the availability of labor in the damaged region.

| **Class** | **Diameter from: [in.]** | **Diameter to: [in.]** | **# Fixed breaks per day per worker** | **# Fixed leaks per day per worker** | **# Available workers** | **Priority** |
|---|---|---|---|---|---|---|
| a | 60 | 300 | 0.33 | 0.66 | User-specified | 1 (highest) |
| b | 36 | 60 | 0.33 | 0.66 | User-specified | 2 |
| c | 20 | 36 | 0.33 | 0.66 | User-specified | 3 |

| | | | | | | |
|---|---|---|---|---|---|---|
| d | 12 | 20 | 0.50 | 1.0 | User-specified | 4 |
| e | 8 | 12 | 0.50 | 1.0 | User-specified | 5 (lowest) |
| u | Unknown or for default diameter data analysis | | 0.50 | 1.0 | User-specified | 6 (lowest) |

Table 37. Restoration function, drainpipes, earthquake hazard

#### 4.7.4.3 Infrastructure improvements

The resistance of a drainage system to earthquake motions can be improved by several methods, as shown in the following bullet list and in Figure 101. For the most critical component, drainpipes, replacement of pipes and joints can improve deformability and sustainability and is a fundamental and effective engineering improvement for existing drainpipes (EPA 2018 & FEMA/NIBS 2005). Quality assurance (QA) for pipe installation constitutes testing and construction inspection. Based on engineering judgment, it is assumed that the cost of standard QA is about 10% of the project budget, and the higher-level QA cost is approximately 15% of the total budget. Therefore, for better QA of retrofit work, the higher-level QA approach is considered in this report, as shown in the last bullet, and the standard QA is assumed to have been applied to the existing components.

Improvements to increase the earthquake resistance of drainage systems include:

- Replace the existing brittle pipes with the ductile pipes.
- Upgrade the existing nonflexible joints with the flexible/expandable joints.
- Assess geotechnical components and strengthen them if they are seismically deficient.
- Apply soil improvements and densification by several measures (e.g., cement mixing and soil compaction).
- Perform routine and regular maintenance for all components and fix any observed problems.
- Conduct the higher-level QA protocol (e.g., daily testing and continual and detailed field inspection).

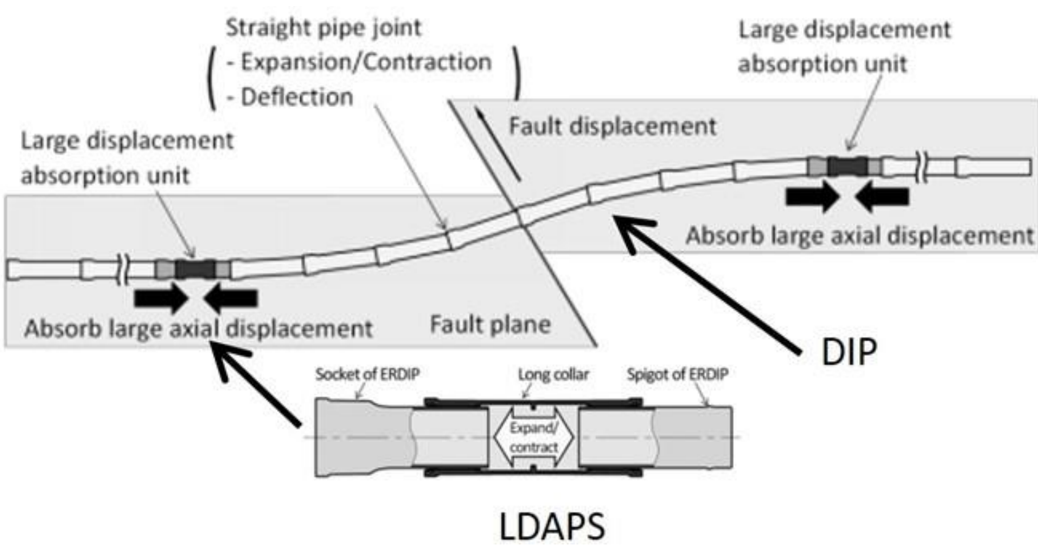



Figure 101. Example of ductile iron pipe (DIP) and special joints

#### 4.7.4.4 Cost-benefit considerations

FEMA (2013a) proposes seismic damage rates for buried pipes that are constructed in the United States. Table 38 (FEMA 2013a) shows the two sets of damage rates: one for brittle pipes and one for ductile pipes that have higher seismic capacity. These rates are based on a certain level of earthquake motion. Therefore, in this report, it is assumed that the conditional pipe damage rates from an earthquake equivalently represent the damage probability for this component and are used to estimate the resiliency index of drainpipes. For pipe damage from seismic shaking, the damage rate (i.e., damage probability) is 1.0 for existing drainpipes and is 0.3 for improved drainpipes.

The cost to improve the resistance to earthquake shaking (i.e., pipe and joint replacement) depends on the current site complexity and environmental situation, but in this report, it is assumed to be 100% of the pipe replacement cost. The total improvement cost, including the higher-level QA, can then be estimated at 105% of the total cost of drainpipe construction. The damage rate is significantly reduced by implementation of the improvement measures, and the recovery process for the drainage system would be substantially faster.

| | **PGV algorithm** | | **PGD algorithm** | |
|---|---|---|---|---|
| | **RR = 0.0001 x PGV$^{(2.25)}$** | | **RR = Prob[liq] x PGD$^{(0.56)}$** | |
| Pipe type | Multiplier | Example of pipe | Multiplier | Example of pipe |
| Brittle sewers/interceptors | 1 | Clay, concrete | 1 | Clay, concrete |
| Ductile sewers/interceptors | 0.3 | Plastic | 0.3 | Plastic |

Table 38. Seismic damage rates of different pipes (FEMA 2013a)

### *4.7.5 Liquefaction hazard*

#### 4.7.5.1 General

Underground drainpipes can also be damaged by large permanent ground deformation (PGD) such as horizontally forced displacement and lateral spreading of soil due to earthquake-induced liquefaction (FEMA 2013a). Drainpipe networks that are buried in liquefiable and weak soils in high-seismicity areas are particularly vulnerable to such liquefaction damage.

#### 4.7.5.2 Key metric for consideration

The intensity threshold that damages a buried drainpipe is assumed to be a PGD of 12 in. (FEMA 2013a, Ferritto 1997, Urayasu 2012, & Nakashima et al. 2015), and the damage state of more than extensive/severe damage is adopted to represent the resiliency index, calculated by damage probability. Liquefaction can result in physical damage and functional loss of a drainage system, and because the failure usually occurs below ground, the cost of geotechnical repair and restoration of a pipe network is high.

#### 4.7.5.3 Infrastructure improvements

Liquefaction resistance for a drainage system can be improved by several measures, including the methods in the following bullet list. Soil improvement greatly raises the capacity of buried piping, the most critical component, to resist liquefaction, and it is an effective engineering improvement for existing buried drainpipes (EPA 2018 & Andrus and Chung 1995). Soil improvement by permeation grouting or by jet grouting strengthens weak soils. As stated in Section 4.7.4.3, it is assumed that the standard-QA cost is about 10% of the project budget, and the higher-level QA cost is approximately 15% of the total budget for a sewage network. Therefore, for better QA of retrofit work, the higher-level QA approach is considered in this report, and the standard QA is assumed to have been applied to the existing components.

Improvements to increase liquefaction resistance for drainage systems include:

- Apply soil improvements and densification by several measures (e.g., cement mixing and soil compaction); see Figure 102.
- Move the pipe network into non-liquefiable areas.
- Replace existing brittle pipes with ductile pipes.

- Upgrade the existing nonflexible joints with the flexible/expandable joints.
- Assess geotechnical components and strengthen them if they are seismically deficient.
- Perform routine and regular maintenance for all components and fix any observed problems.
- Conduct the higher-level QA protocol (e.g., daily testing and continual and detailed field inspection).

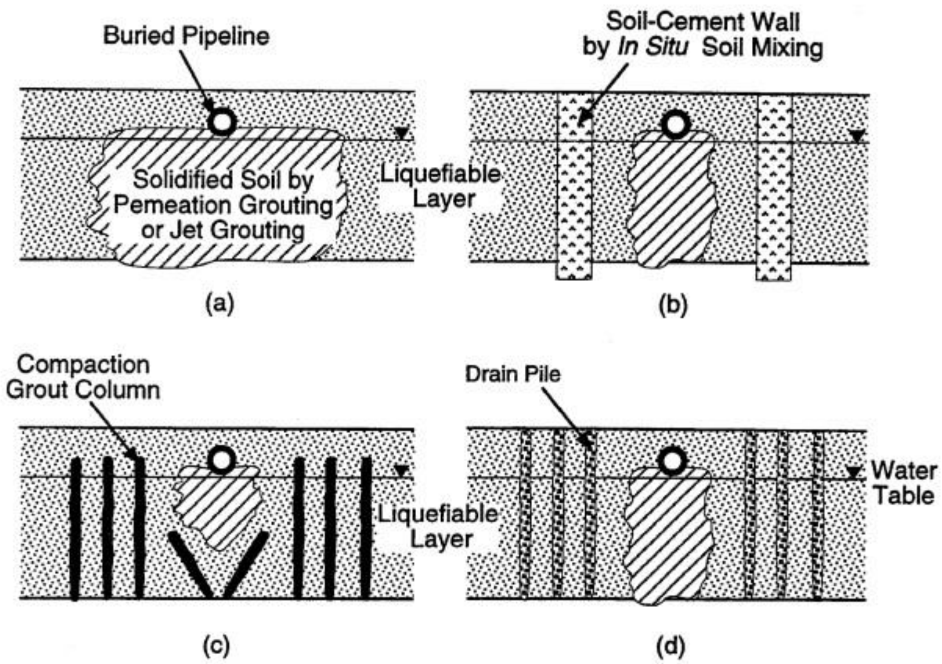



Figure 102. Examples of soil improvement

### 4.7.5.4 Cost-benefit considerations

As shown in Table 38 in Section 4.7.4.4 FEMA (2013a) proposes seismic damage rates for buried pipes. Two sets of damage rates are provided: one for brittle pipes and one for ductile pipes. These rates are based on a certain level of earthquake motion. In this report, it is assumed that the conditional pipe damage rates from an earthquake equivalently represent the damage probability for this component caused on earthquake-induced liquefaction, and the difference in damage between brittle and ductile pipes equals the damage variation between liquefiable and improved soil. Therefore, the damage rates in Table 38 are applied as the damage probability (i.e., resiliency index) for drainpipes from liquefaction. For damage that is caused by liquefaction displacement, the damage rate (i.e., damage probability) is 1.0 for existing drainage systems and is 0.3 for improved systems.

The cost for a liquefaction retrofit (i.e., soil improvement/compaction) highly depends on local constructability, site complexity, and the environmental situation, but in this report, it is estimated at approximately 50% of the pipe replacement cost. The total improvement cost, including the higher-level QA, can then be estimated as 55% of the total cost of drainpipe construction. The damage rate is significantly reduced by implementation of improvement measures. The drainpipe damage level will also likely be lower during an earthquake, and the recovery process for the drainage system will presumably be faster.

### *4.7.6 Wind hazard*

#### 4.7.6.1 General

Because they are typically installed on or below the ground, the components of a drainage system are not especially vulnerable to strong wind hazards such as windstorms and hurricanes. Therefore, in this report, it is assumed that no serious damage to a drainage system would be caused by a wind hazard.

#### 4.7.6.2 Key metric for consideration

This topic does not apply.

#### 4.7.6.3 Infrastructure improvements

This topic does not apply.

#### 4.7.6.4 Cost-benefit considerations

This topic does not apply.

### *4.7.7 Flood hazard*

#### 4.7.7.1 General

Because the components of a drainage system are generally placed on or below the ground, it can be assumed that typical drainage systems will not be physically damaged by disastrous flooding or serious inundation depth. Ground-level components could suffer some operational loss that affects the drainage system's functionality. However, it is expected that this operational loss would be caused by temporary overcapacity due to massive amounts of water and not by serious physical damage. To avoid this potential operational loss, periodic maintenance and cleaning of catch basins and manholes are effective in facilitating proper drainage of rainwater or floodwater. In this report, therefore, it is assumed that drainage systems are not susceptible to flood disasters.

#### 4.7.7.2 Key metric for consideration

This topic does not apply.

#### 4.7.7.3 Infrastructure improvements

This topic does not apply.

#### 4.7.7.4 Cost-benefit considerations

This topic does not apply.

## 5. ROADWAY INFRASTRUCTURE

## 5.1 Highways (on grade)

### *5.1.1 Overview*

Highways are major transportation arteries and corridors that connect various cities and that feed traffic to local streets. Figure 103 shows an example.


Figure 103. Highway

### *5.1.2 Summary*

The results from a literature review of highways are summarized in Table 39 and are presented in the "*Improvement of infrastructure resilience evaluation matrix for selected natural hazards*" table of the project final report. The following subsections of this background report provide more detailed descriptions and background information. On-grade highways are not susceptible to damage from direct wind, thus wind hazard for highways is not discussed in this report.[§] Hurricane-induced flooding is not mentioned specifically but is encompassed in the flood hazard section (Section 5.1.7).

| Hazard | Susceptible | Improvements | Damage index | | Approximate improvement cost as % of cost |
|---|---|---|---|---|---|
| | | | As-is | Improved | |
| **Earthquake** | Y | Geogrid reinforcement of embankment | 0.10 | 0.05 | 10% |
| **Liquefaction** | Y | Soil improvement | 0.10 | 0.05 | 5% |
| **Wind** | N | -- | -- | -- | -- |
| **Flood** | N | -- | -- | -- | -- |
| **Landslide** | Y | Retaining walls | 0.20 | 0.02 | 10% |

Table 39. Summary of findings for highways

[§] Other (elevated) highway components such as sign structures would require evaluation for wind loading.

### *5.1.3 Vulnerability to natural hazards*

Highways have been significantly damaged in past natural hazard events. Figure 104 shows a few examples.

Earthquake (Alaska, USA, 2018)

Liquefaction (Alaska, USA, 1964)

Landslide (Wyoming, USA, 2011)

Hazard does not apply

Flood

Figure 104. Damage to highways from natural hazards

### *5.1.4 Earthquake hazard*

#### 5.1.4.1 General

Highways are susceptible to damage from peak ground displacement. Most of the past highway damage from earthquakes has been the result of poorly compacted underlying materials. The damage states are defined as listed in Table 40 (FEMA 2013a).

| Damage state | Definition | Restoration, days (median) |
|---|---|---|
| DS0 (none) | -- | -- |
| DS1 (minor) | Slight settlement (<100 mm) or offset of the ground. | 1 |
| DS2 (moderate) | Moderate settlement (<300 mm) or offset of the ground. | 2 |
| DS3 (extensive) | Major settlement of the ground (0.5 m or more) | >21 |
| DS4 (complete) | Major settlement of the ground (0.5 m or more) | >21 |

Table 40. Earthquake damage states for highways

### 5.1.4.2 Key metric for consideration

For main highways, the time to open the road and to restore its function after an earthquake is the key metric. The timeline of restoration (FEMA 2013a) for highways is presented in Figure 105.**

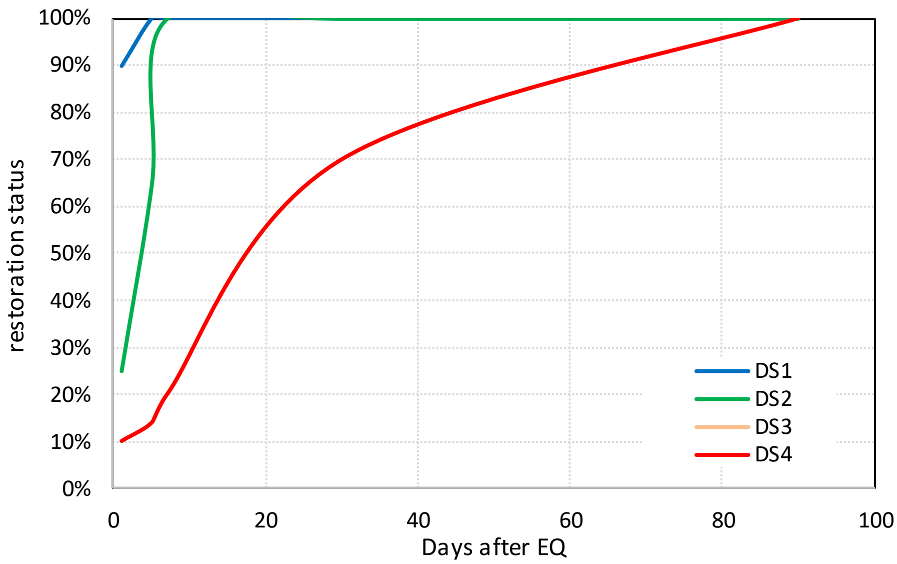



Figure 105. Restoration curve, highways, earthquake hazard

### 5.1.4.3 Infrastructure improvements

Retrofit of existing roadways is too expensive. Instead, a rapid repair of pavement procedure could be developed after an earthquake that damages on-grade highways. For new highway construction, earthquake performance can be improved by several methods. Examples include (FEMA 2006):

- Use a two-tier design approach: fully operational for a 100-year earthquake and no major damage for a 1,000-year event.
- Properly compact the underlying embankment.
- Use earthquake-resistant embankments or foundations.
- Perform a check and ensure slope stability of the embankment.
- Use geogrid reinforcement at the toe of the pavement; see Figure 106.

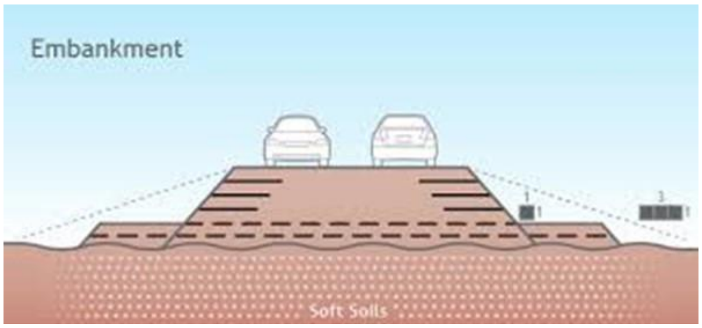



Figure 106. Geogrid reinforcement

** DS3 and DS4 are overlapped.

### 5.1.4.4 Cost-benefit considerations

FEMA (2013a) provides fragility functions for highways in the United States. In this report, the fragility parameters are used for the reinforced embankment case and are modified for the unreinforced embankment case to account for the lower expected quality in worldwide application. Figure 107 presents the fragility functions for unreinforced and reinforced embankments, respectively. Note that for an earthquake with a PGD of 500 mm (20 in.), the probability of exceeding DS3/DS4 is 10% for unreinforced embankments and is 5% for reinforced embankments. In other words, by using seismic components, at a minimal cost of 10%, the probability that restoration will take 3 weeks is reduced by a factor of 2 (Barry 2014). Such expedited restoration provides significant economic benefits and helps the community with recovery.

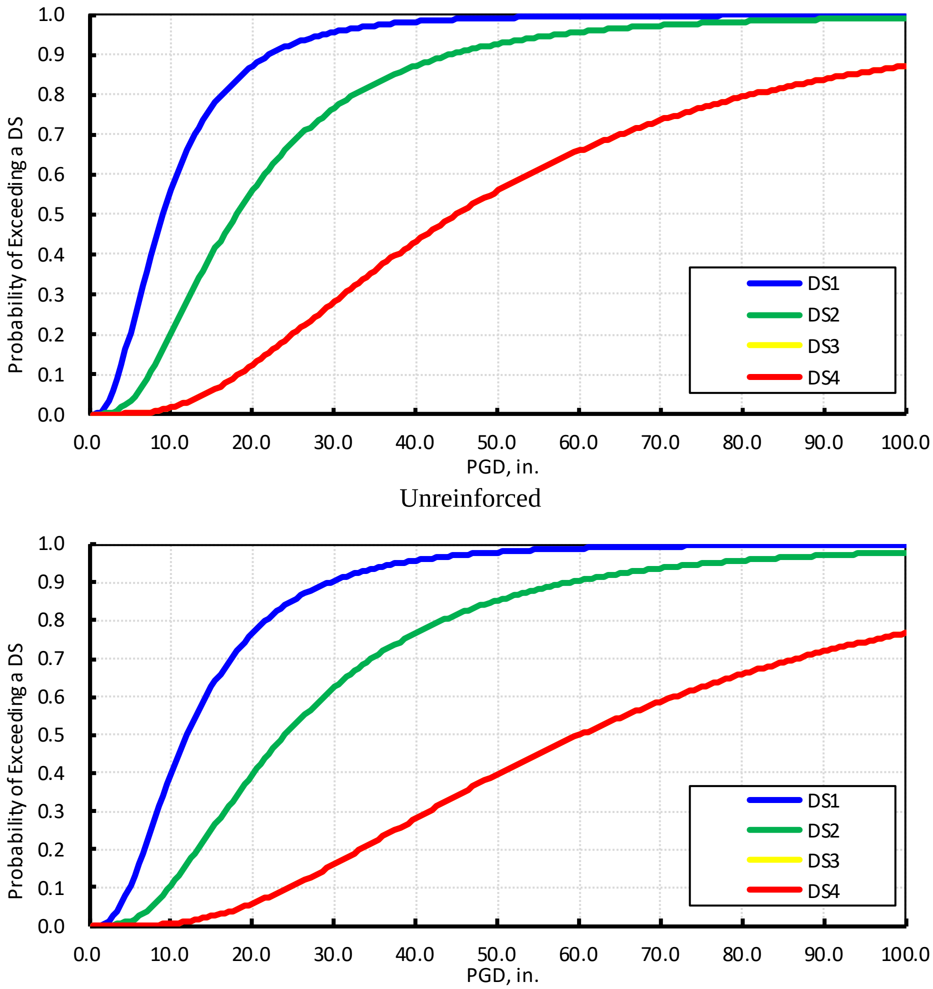



Figure 107. Damage fragility functions for highways

## *5.1.5 Liquefaction hazard*

### 5.1.5.1 General

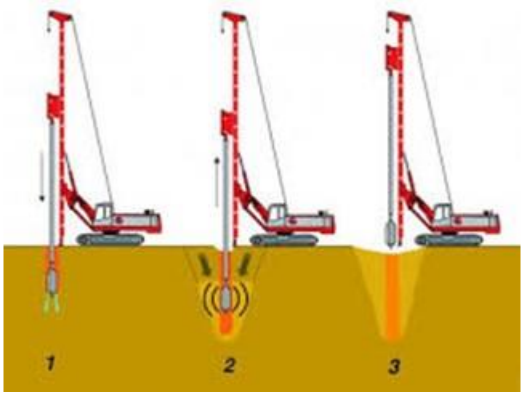
Roadways that are constructed on weak soils in earthquake zones are vulnerable to damage from liquefaction.

### 5.1.5.2 Key metric for consideration

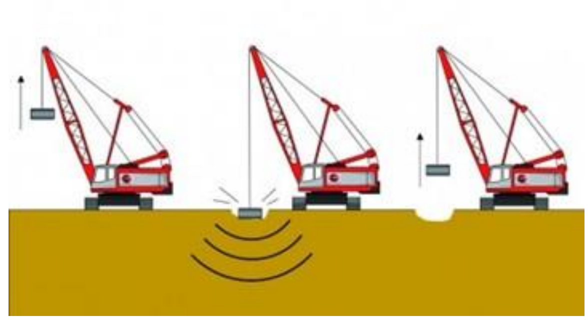
Liquefaction can damage highway embankments and result in loss of functionality. The time that is required to restore operation is a key metric to consider. The loss of highway operation can have severe consequences economically and can impede the supply delivery chain.

### 5.1.5.3 Infrastructure improvements

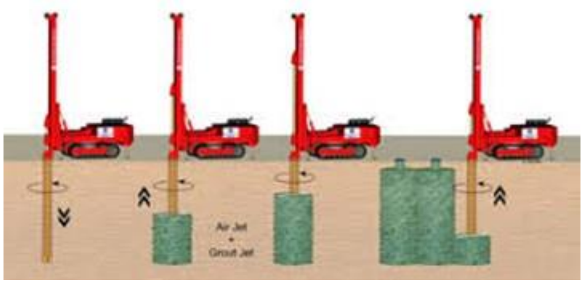
Retrofit of existing roadways on liquefiable soil is not cost-effective. This section of the report presents methods for new construction. For highways, liquefaction performance can be improved by several methods that improve the quality of the underlying soil. Figure 108 shows examples. The methods that are presented have the lowest associated costs.



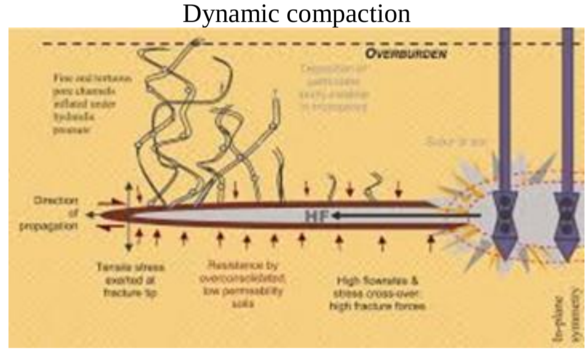

Vibro compation



Dynamic compaction




Grouting




Pressure-injected lime slurry

Figure 108. Examples of geotechnical liquefaction mitigation

### 5.1.5.4 Cost-benefit considerations

For highways, the benefit of liquefaction-resistance improvement is similar to that for earthquake hazard improvement; damage is reduced from 10% for existing highways to 5% for highways on improved soils. The initial soil improvement cost is estimated to be 5%, which is based on the assumption that only a limited length of a long highway is subject to liquefaction.

## *5.1.6 Landslide*

### 5.1.6.1 General

Highways in mountainous regions are susceptible to damage and to service interruption that can last months because of landslides.

### 5.1.6.2 Key metric for consideration

Structural reliability is used to assess the vulnerability of on-grade highway components. Structural reliability is the probability that the structure will not reach a limit state (e.g., a failure state) during a given period. With this approach, the responses of various on-grade highway components are averaged out, thus it eliminates some of the uncertainties.

The probability of failure is related to the factor of safety that is used in the design; a higher factor of safety implies a lower probability of failure from landslides.

#### 5.1.6.3 Infrastructure improvements

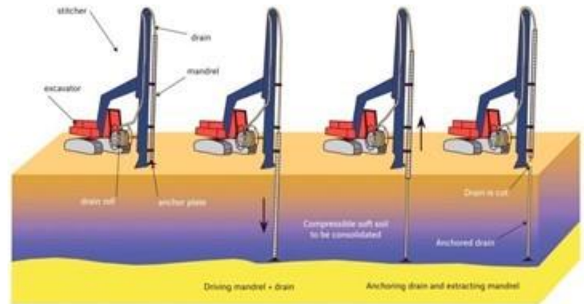
For highways, mitigation against landslides can be improved by several methods. Examples are outlined in the following list and are shown in Figure 109:

- Geometric (slope) reconfiguration and removal of unstable soil
- Mechanical/structural solutions:
    - Provide tiebacks or soil nails
    - Shotcrete the surface
    - Construct retaining walls and steel-wood walls
- Hydrological solutions:
    - Add drainage to reduce water pressure
    - Prevent water from entering the hillside by diversion
- Erosion control:
    - Add steel netting
    - Add geomats
    - Add coconut fiber mesh

Steel netting

Soil nails, tieback anchors

Surface shotcretre




Vertical drains

Figure 109. Examples of landslide mitigation

#### 5.1.6.4 Cost-benefit considerations

The use of cantilever concrete retaining walls is the most common method to mitigate landslide effects. The additional cost of the retaining walls is assumed to be 10% of the cost of construction, based on the assumption that only a limited section of the highway requires protection. As previously mentioned, landslides have resulted in the shutdown of highways for months. Therefore, in this report, it is assumed that mitigation will reduce the risk of loss of operation and the risk of damage by a factor of 10; risk is assumed to decrease from 20% for existing highways to 2% for improved highways.

### *5.1.7 Flood hazard*

#### 5.1.7.1 General

Highway roads are sometimes shut down because of severe flooding and deep inundation. When shutdown occurs, the community and residents lose a transport function and mobility for a certain period. However, on-grade roads of main highways generally do not incur serious physical damage from flooding or inundation. Severe damage to road segments can sometimes be caused by a flood-induced landslide, which is discussed in Section 5.1.6. Consequently, in this report, it is assumed that the on-grade road of a main highway is not susceptible to a flood disaster.

#### 5.1.7.2 Key metric for consideration

This topic does not apply.

#### 5.1.7.3 Infrastructure improvements

This topic does not apply.

#### 5.1.7.4 Cost-benefit considerations

This topic does not apply.

## 5.2 Highway bridges

### *5.2.1 Overview*

Highway bridges are a means to overcome road obstacles (e.g., rivers, railroad tracks, roadways, and other highways) or serve as connectors or ramps to allow traffic to flow from one highway to another. Figure 110 shows an example.

Figure 110. Highway bridges

### *5.2.2 Summary*

The results from a literature review of highway bridges are summarized in Table 41 and are presented in the "*Improvement of infrastructure resilience evaluation matrix for selected natural hazards*" table of the project final report. The following subsections of this background report provide more detailed descriptions and background information. Hurricane-induced flooding is not mentioned specifically but is encompassed in the flood hazard section (Section 5.2.7).

| Hazard | Susceptible | Improvements | Damage index | | Approximate improvement cost as % of cost |
|---|---|---|---|---|---|
| | | | As-is | Improved | |
| **Earthquake** | Y | Seismic design for bridge and components | 0.4 | 0.05 | 10% |
| **Liquefaction** | Y | Pile foundations | 0.3 | 0.05 | 20% |
| **Wind** | Y | Fatigue-resistant detailing for steel bridges | 0.05 | 0.01 | 5% |
| **Flood** | Y | Scour mitigation, riprap, etc. | 0.05 | 0.02 | 5% |
| **Landslide** | Y | Soil improvements | 0.5 | 0.16 | 15% |

Table 41. Summary of findings for highway bridges

### *5.2.3 Vulnerability to natural hazards*

Highway bridges have been significantly damaged in past natural hazard events. Figure 111 shows examples.

Earthquake (California, USA, 1994)

Liquefaction (Japan, 1994)

Wind (Italy, 2018)

Flood (Arizona, USA, 2017)

Landslide (California, USA, 2017)

Figure 111. Damage to highway bridges from natural hazards

### *5.2.4 Earthquake hazard*

#### 5.2.4.1 General

Highway bridges are susceptible to damage from peak ground acceleration (PGA). Most of the past highway bridge damage from earthquakes has been the result of poorly designed reinforced-concrete members and steel connections. The damage states are defined as listed in Table 42 (FEMA 2013a).

| Damage state | Definition | Restoration, days (median) |
|---|---|---|
| DS0 (none) | -- | -- |
| DS1 (minor) | Minor cracking and spalling to the abutment, cracks in shear keys at abutments, minor spalling and cracks at hinges, minor spalling at the column (damage requires no more than cosmetic repair) and/or minor cracking to the deck | 1 |
| DS2 (moderate) | Any column experiencing moderate (shear cracks)<br>Cracking and spalling (column structurally still sound), moderate movement of the abutment (<2”), extensive cracking and spalling of shear keys, any connection having cracked shear keys or bent bolts, keeper bar failure without unseating, rocker bearing failure and/or moderate settlement of the approach. | 3 |
| DS3 (extensive) | Any column degrading without collapse – shear failure - (column structurally unsafe), significant residual movement at connections, and/or major settlement approach, vertical offset of the abutment, differential settlement at connections, shear key failure at abutments. | 75 |
| DS4 (complete) | Any column collapsing and connection losing all bearing support, which may lead to imminent deck collapse, tilting of substructure due to foundation failure. | 230 |

Table 42. Earthquake damage states for highway bridges

### 5.2.4.2 Key metric for consideration

A key consideration for highway bridges is the amount of time that it takes to restore operations after an earthquake. Figure 112 presents the timeline of restoration (FEMA 2013a) for highway bridges.

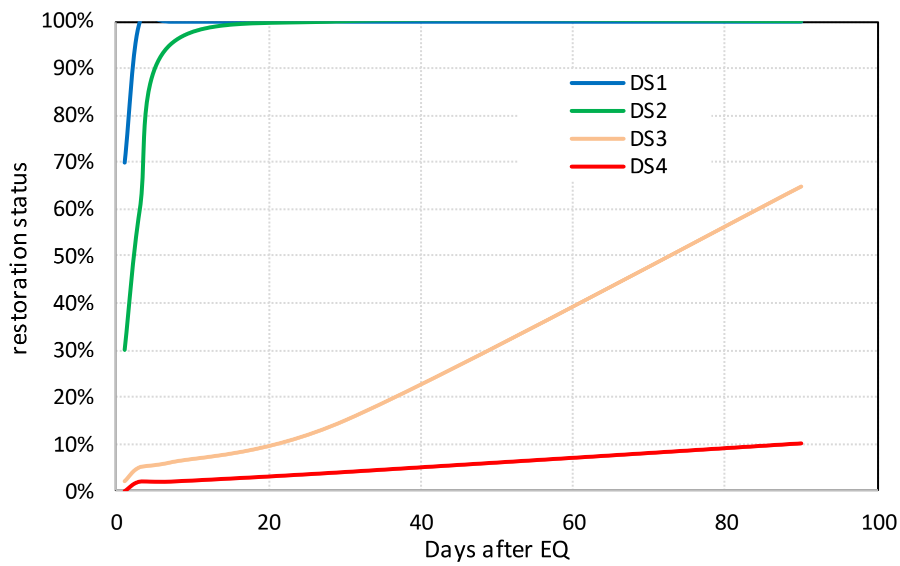



Figure 112. Restoration curve, highway bridges, earthquake hazard

### 5.2.4.3 Infrastructure improvements

Retrofit of existing highway bridges usually involves upgrading the bridge to DS2 or DS3 to ensure that it does not collapse in an earthquake. For new bridges, earthquake performance can be improved by several methods. The following list (Barry 2014) outlines examples:

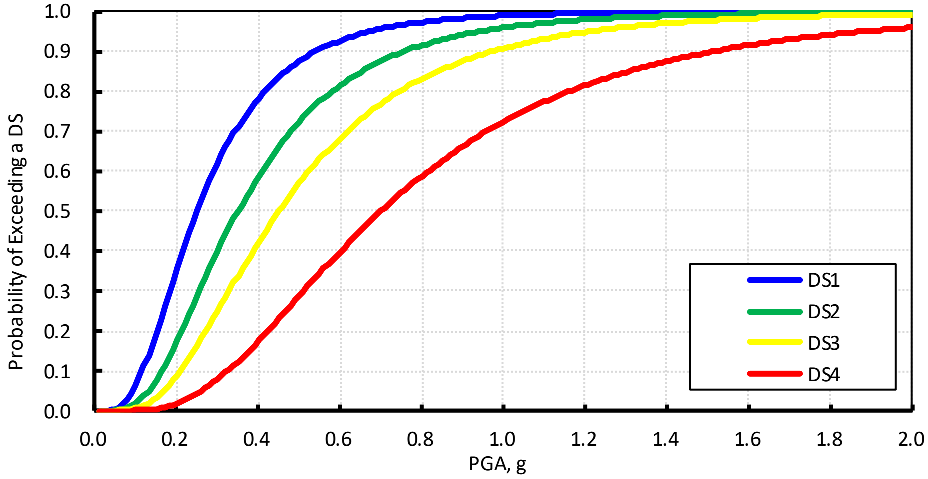
- Follow the current highway bridge seismic design procedure that is used in California or in Japan.

- Ensure that the concrete or steel superstructure and pier caps remain elastic during earthquakes by using a capacity design concept.
- Ensure that the foundation remains elastic during earthquakes by using capacity design.
- Design the column elements to act as the yielding elements (fuses) to dissipate seismic energy.
- Detail the columns to be ductile by providing closely spaced lateral reinforcement; see Figure 113.
- Design in-span hinges with sufficient width to prevent unseating of the bridge even during very large earthquakes.
- During construction, perform regular inspections, including during concrete pours.
- Conduct random sampling and materials testing.
- For steel components, require submittal of a welding plan and inspect welding.

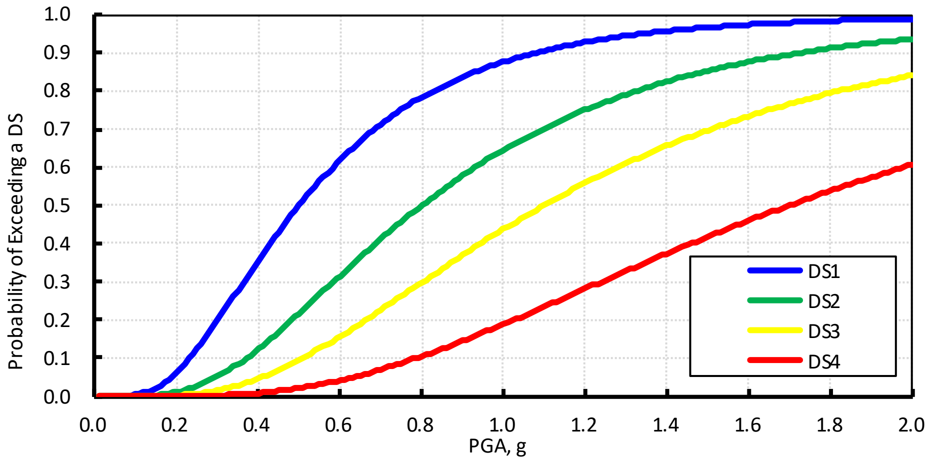
Figure 113. Ductile detailing of concrete columns

#### 5.2.4.4 Cost-benefit considerations

FEMA (2013a) provides fragility functions for highway bridges in the United States. In this report, the multi-span pre-stressed concrete girder superstructure (types 17–19 in FEMA 2013a) is used as a reference, because this type of bridge is currently the most common new bridge that is being constructed. In addition, the findings for this bridge type are similar for other bridges. Figure 114 presents the fragility functions for bridges with a conventional design and a seismic design, respectively. Note that for a $M_w$ 7 earthquake with a likely PGA of 0.4g, the probability of exceeding DS3 is 40% for the conventional design and is 5% for the seismic design. In other words, by using seismic components at an additional estimated cost of 10%, the probability that the bridge will be nonoperational for 2.5 months decreases from 40% to 5%. Such expedited restoration provides significant economic benefits, reduces downtime and repair costs, and helps with the community recovery.




Conventional design




Seismic design

Figure 114. Damage fragility functions for highway bridges, earthquake hazard

### *5.2.5 Liquefaction hazard*

#### 5.2.5.1 General

Bridge piers that constructed on weak soils in earthquake zones are vulnerable to damage from liquefaction.

#### 5.2.5.2 Key metric for consideration

Liquefaction can cause loss of support due to large ground deformation, resulting in highway bridge damage or collapse of the entire bridge.

#### 5.2.5.3 Infrastructure improvements

Retrofit of existing highway bridges on liquefiable soil is similar to new construction and involves enlarging the foundation and adding piles. This section therefore presents methods for new construction. For highway bridges, liquefaction performance can be improved by using a deep foundation. Figure 115 shows examples.

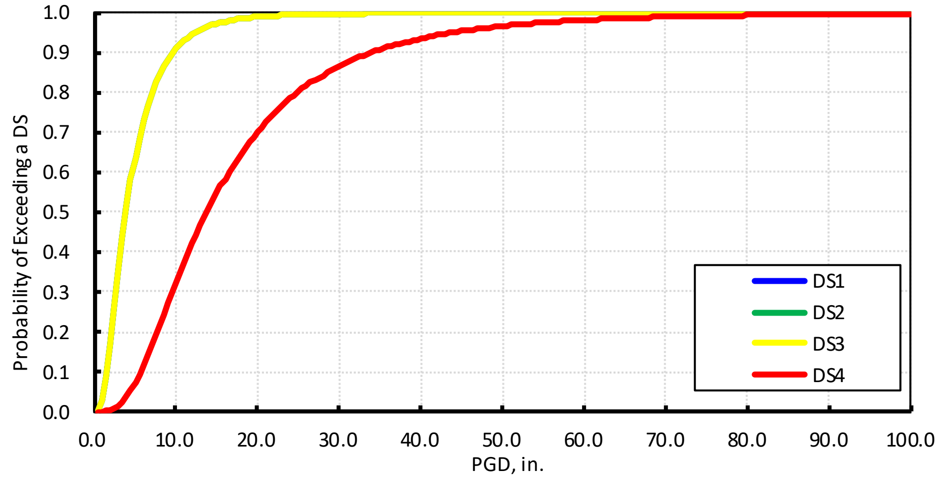

Steel H driven piles | Prestressed driven piles

Cast-in-drilled-hole (CIDH) pile | Cast-in-steel-shell (CISS) pile

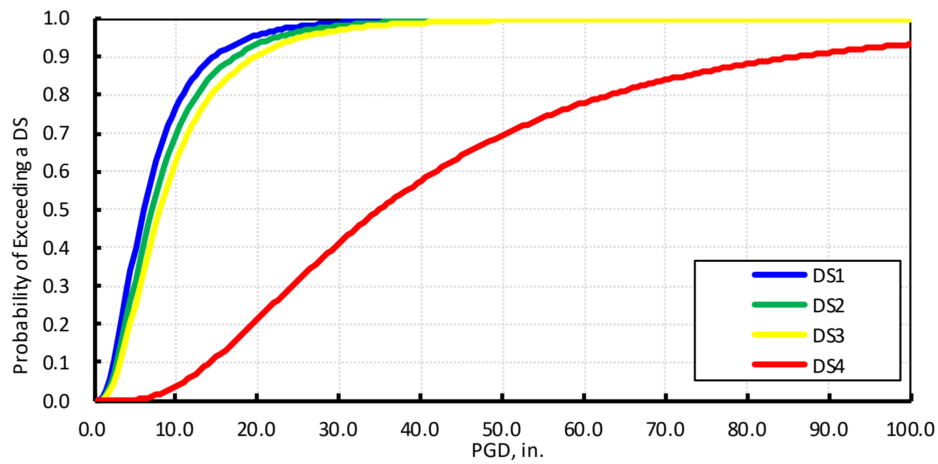
Figure 115. Examples of deep foundations

### 5.2.5.4 Cost-benefit considerations

FEMA (2013a) provides fragility functions for highway bridges in the United States. In this report, the multi-span pre-stressed concrete girder superstructure (types 17–19 in FEMA 2013a) are used as a reference, because this type of bridge is currently the most common new bridge that is being constructed. In addition, the findings for this bridge type are similar for other bridges. Figure 116 presents the fragility functions for bridges with a conventional foundation and with a deep foundation (derived assuming proper deep foundation seismic design), respectively. Note that for a PGD of 10 in., the probability of exceeding DS4 is 30% for the conventional design and is 5% for the deep foundation seismic design. In other words, by using seismic components at an additional estimated cost of 20%, the probability that the bridge will be nonoperational for 8 months decreases from 30% to 5%. Such expedited restoration provides significant economic benefits, reduces downtime and repair costs, and helps with community recovery.




Conventional design




Seismic design

Figure 116. Damage fragility functions for highway bridges, liquefaction hazard

### *5.2.6 Wind*

#### 5.2.6.1 General

Concrete highway bridges are heavy and stiff and thus are less susceptible to damage from wind loading. In contrast, because of their relative flexibility, steel bridges can be susceptible to wind forces. Steel bridges respond to both crosswind and along-wind forces. The spectacular collapse of the Tacoma Narrows Bridge in 1940 is an example of crosswind failure. Today, bridges are constructed to have sufficient torsional rigidity to mitigate this type of failure. Bridges are subject to stress reversal at connections and at members because of wind loading. Thus, if these elements are not adequately designed, high-cycle, low-amplitude fatigue can cause them to fail from along-wind loading at stresses that are much lower than their nominal capacity.

#### 5.2.6.2 Key metric for consideration

The detailing of welded connections is the most critical element in determining the fatigue life of a highway bridge.

### 5.2.6.3 Infrastructure improvements

For steel bridges, the design life should include connection details that have a long ("infinite") fatigue life; see Figure 117. Other examples of improvements include:

- Use a longer design life. Many bridges are designed for a 50- to 75-year design life, but in practice remain in service for a much longer period.
- Use a conservative design because bridge loading tends to increase over time as the vehicle payloads increase.
- Use a wind speed that is larger than the design wind speed.
- Avid the use of eyebars in the design.
- Do not weld stiffeners to the bottom (tension) flanges.
- Perform regular and random inspection of the bridge and repair anomalies.
- Use redundant details and connections that have a longer fatigue life.

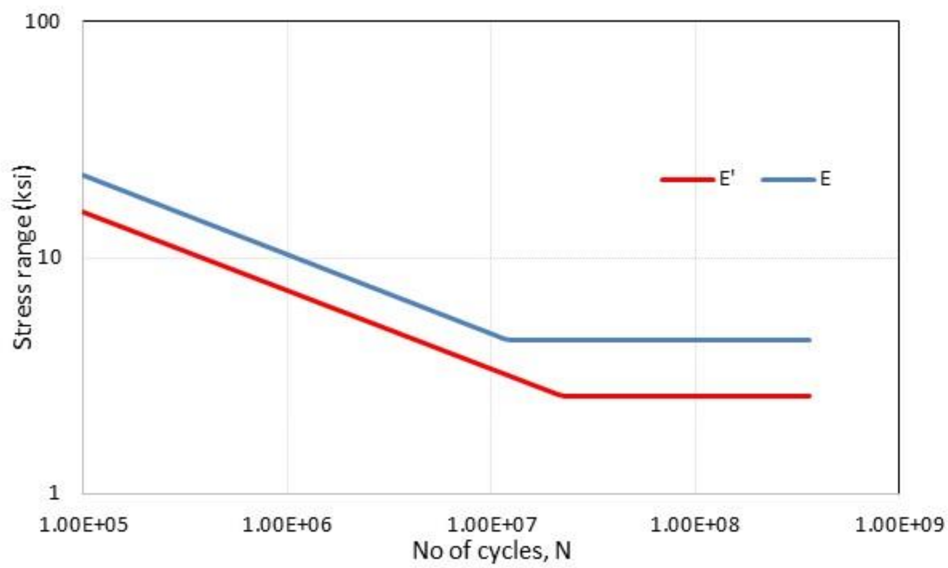



Figure 117. Examples of wind mitigation for fatigue by improved connections

### 5.2.6.4 Cost-benefit considerations

The use of better detailing and connections is the most efficient way to design for fatigue. As Figure 117 (AASHTO 2017) shows, by marginally improving the connection detailing (from ASSHTO category E′ to category E), at a stress range of 6 ksi (42MPa, expected for normal applications), the bridge component can withstand 4 times as many stress reversal cycles from wind loading.

The cost for such improvements is a fraction of the total replacement cost of a highway bridge and is assumed to be 5% in this report. Also, the likelihood of failure for the design life of the bridge is assumed to be reduced, from 5% for existing bridges to 1% for bridges with improved connections.

### *5.2.7 Flood hazard*

#### 5.2.7.1 General

It is assumed that highway bridges are constructed high enough that there is little chance of overtopping during floods. Highway bridge piers, however, are susceptible to damage from flooding. Vulnerabilities are defined as listed in Table 43 (FEMA 2013c).

| **Overall vulnerability** | **Inundation** | **Scour** | **Hydraulic pressure & debris** | **Financial loss** | **Loss of operation** |
|---|---|---|---|---|---|
| High | Low | Medium | High | High | Medium |

Table 43. Flood vulnerability, highway bridges

#### 5.2.7.2 Key metric for consideration

Figure 118 presents the status of damage (FEMA 2013c) for multi-span highway bridges from flooding. Note that the potential for scour and bridge failure increases exponentially for larger events.

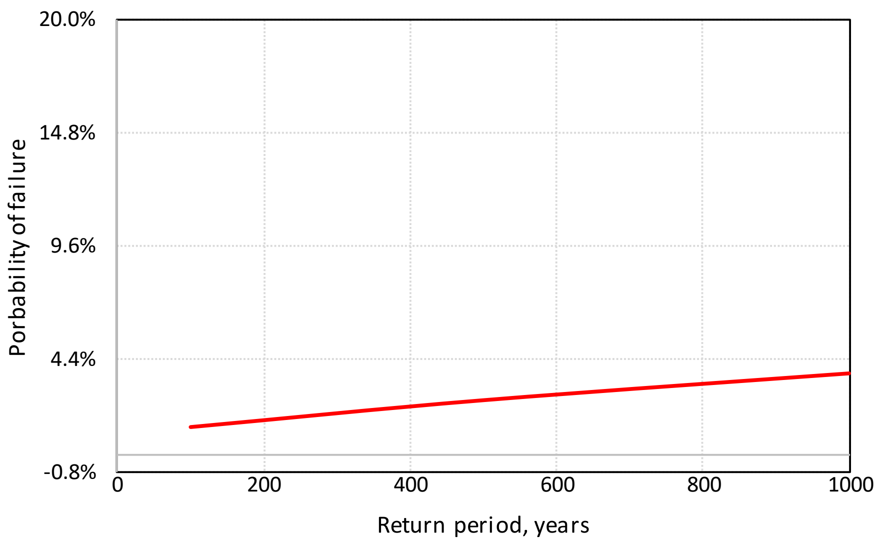



Figure 118. Percentage of damage as a function of inundation depth

#### 5.2.7.3 Infrastructure improvements

For highway bridges, flood performance can be improved by several methods. Examples include:

- Conduct a hydrological study and accurately estimate the flow rate, including all tributaries.
- Align the bridge piers to mitigate scour; for example, use oval instead of rectangular foundations.
- Use large foundations to ensure bridge stability if scour occurs.
- Reduce the number of piers that are in water.
- Use riprap (RSP, or rock slope protection) to protect the bridge piers; see Figure 119 (FHWA 2009).
- Provide barriers in front of the piers to prevent debris from collecting.
- Perform periodic underwater investigation to assess any local scour.

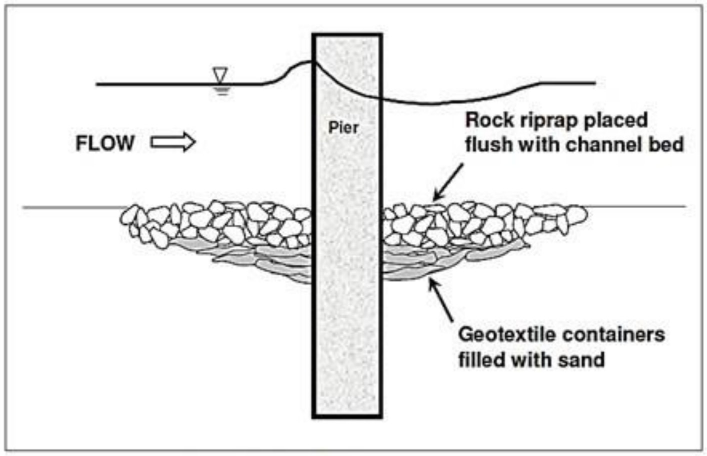



Figure 119. Example of scour mitigation for bridge piers

#### 5.2.7.4 Cost-benefit considerations

For bridge piers, it is assumed that the additional cost of providing scour mitigation is 5% of the total cost of construction. It is also assumed that proper mitigation reduces the probability of failure from that which could occur during a 1,000-year flood to that from a 100-year flood, meaning from 5% to 2%.

### *5.2.8 Landslide*

#### 5.2.8.1 General

The damage to highway bridges that are in landslide-susceptible areas is induced by large permanent ground deformation (PGD), such as horizontally forced displacement due to ground movement (FEMA 2013a). If bridges and soils in those areas have not been designed to account for large displacement and huge soil loading, they are particularly vulnerable to landslide hazards.

#### 5.2.8.2 Key metric for consideration

The intensity threshold that severely damages a highway bridge is assumed to be a PGD of 14 in. for a landslide hazard (FEMA 2013a), and the complete damage state (DS4) is adopted to represent the resiliency index, evaluated by damage probability. A landslide would result in physical damage and functional loss of a bridge and highway network. Because the failure of the bridge and the soil are extensive and the repair work is very difficult and takes long time, the cost of geotechnical restoration and bridge structural repair could be high.

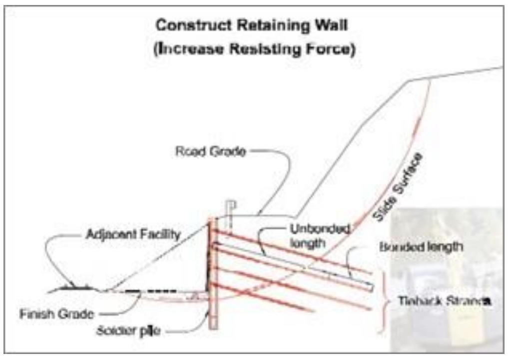
#### 5.2.8.3 Infrastructure improvements

To resist the large permanent ground movements that occur during a landslide, two possible countermeasures for improvements can be considered. One is to improve the soil by using ground reinforcement techniques so that the ground is stable during disasters that can cause landslides. Another countermeasure is to add more capacity and stiffness to the highway bridge so that it can withstand the loading and displacement that the moving soil creates. Because very few highway bridges have collapsed from structural failure of a pier or a foundation in past landslides, and because the most severe damage has been induced by soil failures (FHWA 2006), in this report, soil improvement is considered to be a more cost-effective method and easier to implement than highway bridge structural strengthening is.

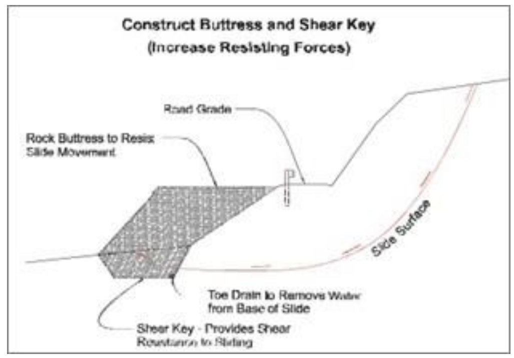
Soil resistance to landslide movement can be improved by several measures, as itemized in the following bullet list. The soil improvement and strengthening that are proposed in the first bullet increase the stability capacity of the soil that surrounds highway bridges (FHWA 2006 & ODOT 2012). This method improves weak soil by increasing its resistance capacity and by reducing the driving force. It is assumed that the standard-QA cost is about 10% of the project budget, and the higher-level QA cost is approximately 15% of the total budget for highway bridge construction. Therefore, for better QA of retrofit work, the higher-level QA approach is considered in this report, as shown in the last bullet, and the standard QA is assumed to have been applied to the existing components.

Improvements to increase landslide resistance for highway bridges include:

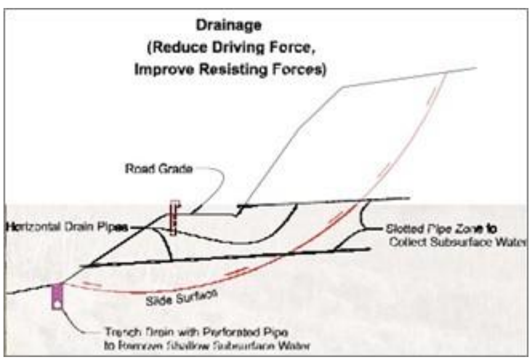
- Apply soil improvement and strengthening by several measures (e.g., retaining walls, buttresses and shear keys, drainage, and a stable slope angle); see Figure 120.
- Strengthen the bridge pier and foundation to resist and to accommodate ground movements.
- Assess geotechnical components and strengthen them if they have landslide deficiencies.
- Perform routine and regular maintenance for all components and fix any observed problems.
- Conduct the higher-level QA protocol (e.g., daily testing and continual and detailed field inspection).



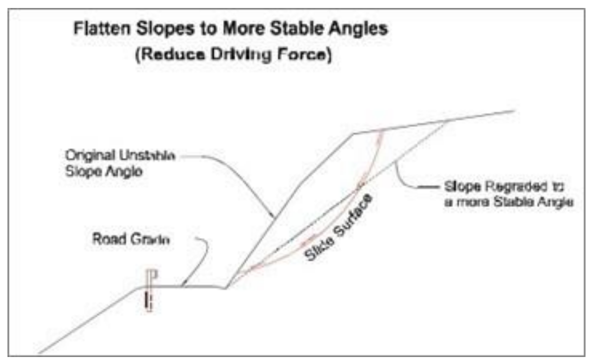

Retaining wall




Buttress and shear key




Drainage




Stable slope angle

Figure 120. Examples of soil improvement for landslide hazards

### 5.2.8.4 Cost-benefit considerations

FEMA (2013a) developed a fragility function for PGD for highway bridges that are built in the United States. Figure 121 shows the fragility curve that expresses the probability of exceeding the complete damage state (DS4) and the horizontal ground displacement. Based on implementation of the improvement methods that are proposed in Section 5.2.8.3, the possible landslide displacement is expected to be greatly reduced. Therefore, it is assumed that the ground displacement due to a certain level of landslide is 14 in. (i.e., the median value of complete damage that is shown in the fragility curve) (FEMA 2013a) before any soil improvements, and that ground displacement after soil improvements is reduced to 7 in. (i.e., reduction by half). The fragility parameters of the function are then applied to estimate the damage probability (i.e., resiliency index) for a highway bridge from a landslide. For displacement with a PGD of 14 in. before any soil improvements, the probability of exceeding the complete damage state (DS4) is 50%. The PGD of 7 in. after implementation of soil improvements results in a lower probability of complete damage, estimated as 16%.

The cost of improvement measures greatly depends on the characteristics, the area, and the volume of the soil and the local ground situation, but in this report, the cost to improve the landslide hazard resistance (i.e., soil improvements) is assumed to be an average of 15% of the replacement cost for highway bridge itself. The total improvement cost, including the higher-level QA, is then estimated as 15% of the total cost of bridge construction. Thus, highway bridge damage due to a landslide is expected to be minor after soil improvements, and the restoration of a highway bridge after a landslide will be completed in less time. Such expedited recovery helps avoid considerable economic loss and benefits the public.

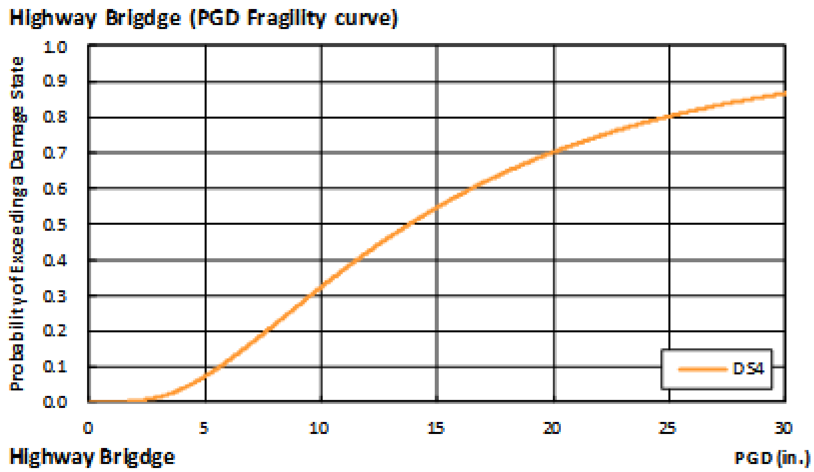



Figure 121. Horizontal displacement fragility functions for highway bridges

## 5.3 Secondary urban roads (on grade)

### *5.3.1 Overview*

Urban roads are the transportation structures that carry city traffic and that feed to highways. Figure 122 shows an example.

Figure 122. Urban road

### *5.3.2 Summary*

The results from a literature review of secondary urban roadways are summarized in Table 44 and are presented in the "*Improvement of infrastructure resilience evaluation matrix for selected natural hazards*" table of the project final report. The following sections of this background report provide more detailed descriptions and background information. On-grade secondary urban roadways are not susceptible to damage from direct wind, thus that hazard is not discussed in this report.[††] Hurricane-induced flooding is not mentioned specifically but is encompassed in the flood hazard section (Section 5.3.6). In addition, because roadways are in urban areas, they are not expected to be affected by landslides.

[††] Other (elevated) roadway components such as sign structures would require evaluation for wind loading.

| Hazard | Susceptible | Improvements | Damage index | | Approximate improvement cost as % of cost |
|---|---|---|---|---|---|
| | | | As-is | Improved | |
| **Earthquake** | Y | Compact the underlying material; use earthquake-resistant foundations | 0.10 | 0.05 | 5% |
| **Liquefaction** | Y | Improve soil | 0.10 | 0.05 | 5% |
| **Wind**[‡‡] | N | -- | -- | -- | -- |
| **Flood** | Y | Provide barriers; improve drainage | 0.10 | 0.05 | 3% |
| **Landslide** | N | -- | -- | -- | -- |

Table 44. Summary of findings for secondary urban roads

### *5.3.3 Vulnerability to natural hazards*

Urban roadways have been significantly damaged in past natural hazard events. Figure 123 shows examples.

Earthquake (California, USA, 2015)

Liquefaction (Chirstchurch, New Zealand, 2011)

Flood Yangon (2018)

Sinkhole triggered by a heavy rain (California, USA)

Figure 123. Damage to secondary urban roads from natural hazards

[‡‡] Hurricanes can cause fallen objects that can results in closure of roads. This secondary impact is not considered in this section.

### *5.3.4 Earthquake hazard*

#### 5.3.4.1 General

Secondary urban roads are susceptible to damage from permanent ground displacement (PGD) that occurs from earthquake shaking. Most of the damage from past earthquakes has been the result of poorly compacted underlying material. The damage states are defined as listed in Table 45 (FEMA 2013a). Note that both complete damage and extensive damage are combined into a single damage state, DS3, for urban roads.

| Damage state | Definition | Restoration, days (mean plus one standard deviation): ATC-13, 1985 |
|---|---|---|
| DS1 | Defined by slight settlement (few inches) or offset of the ground. | 1 |
| DS2 | Defined by moderate settlement (several inches) or offset of the ground. | 4 |
| DS3 | Defined by major settlement of the ground (few feet). | 37 |

Table 45. Earthquake damage states for secondary urban roads

#### 5.3.4.2 Key metric for consideration

A key consideration is the amount of time that it takes to restore secondary urban roads after an earthquake. Figure 124 presents the timeline of restoration (FEMA 2013a) for secondary urban roads.

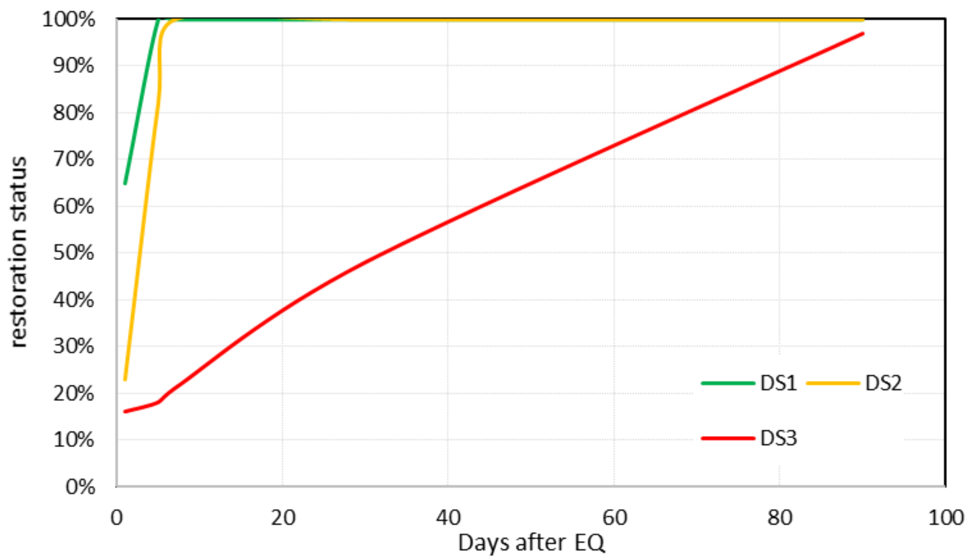



Figure 124. Restoration curve, secondary urban roads, earthquake hazard

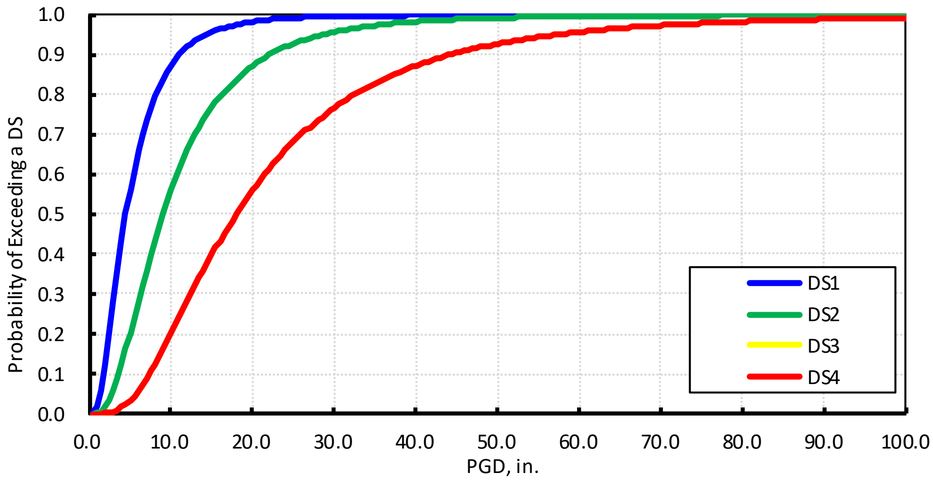
#### 5.3.4.3 Infrastructure improvements

For new construction of urban roadways, earthquake performance can be improved by several methods. Examples include (FEMA 2006):

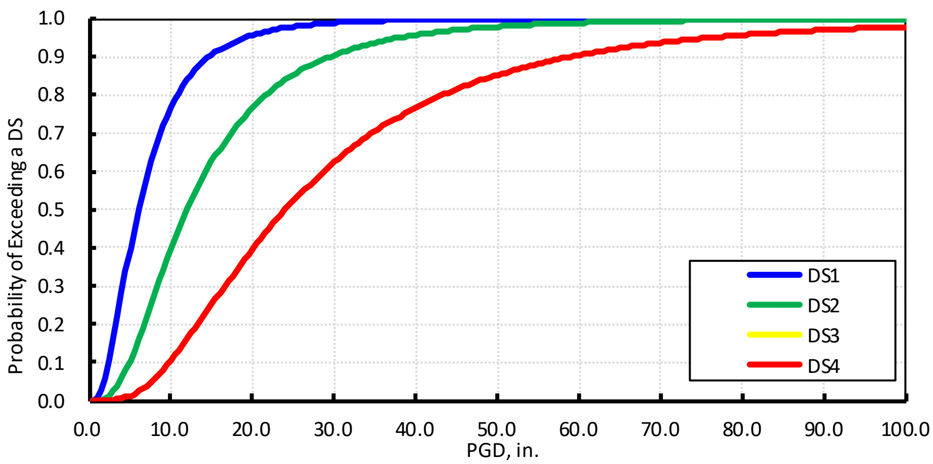
- Use a two-tier design approach that is fully operational for a 100-year earthquake and that has no major damage for a 1,000-year event.
- Properly compact the underlying material.
- Use earthquake-resistant foundations.

### 5.3.4.4 Cost-benefit considerations

FEMA (2013a) provides fragility functions for urban roads in the United States. Figure 125 presents the fragility functions for urban roads. Note that for an earthquake with a PGD of 200 mm (8 in.), the probability of exceeding DS3 is about 2 times greater for the standard design than for the seismic design (10% versus 5%). The repair and restoration time difference is 90 days versus 7 days, respectively. Therefore, by introducing seismic improvements (e.g., removal of old pavement, compaction of the underlying material, and installation of new pavement) at 5% of the replacement cost, the restoration time can be reduced from 90 days to 7 days. Such expedited restoration provides significant economic benefits and helps with the community recovery.




Standard design




Seismic design

Figure 125. Damage fragility functions for secondary urban roads§§

## *5.3.5 Liquefaction hazard*

### 5.3.5.1 General

Urban roadways that are constructed on weak soils in earthquake zones are vulnerable to damage from liquefaction.

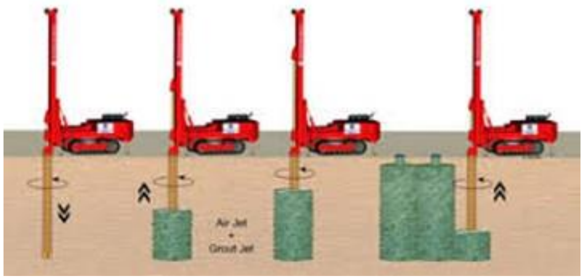
§§ DS3 and DS4 are identical

### 5.3.5.2 Key metric for consideration

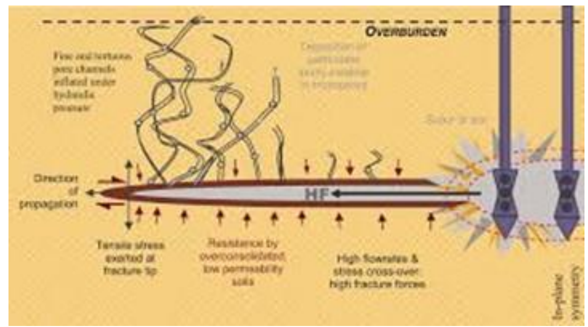
Liquefaction can result in damage and loss of operation of urban roads by large ground deformation.

### 5.3.5.3 Infrastructure improvements

Retrofit of existing urban roadways that are on liquefiable soil is not cost-effective. In many cases, the cost can be significantly higher in cities that have an extensive network of underground metropolitan transportation, gas and water pipelines, and electrical conduits. Therefore, this section presents only methods for new construction. These methods do not produce excessive noise and vibration, both of which are undesirable in urban areas. For urban roads, liquefaction performance can be improved by several methods to improve the quality of the underlying soil. Figure 126 provides some typical examples.




Grouting




Pressure-injected lime slurry

Figure 126. Examples of geotechnical liquefaction mitigation for secondary urban roads

### 5.3.5.4 Cost-benefit considerations

For urban roads, the costs and benefits of liquefaction performance improvement methods are similar to the costs and benefits of improvements against earthquake hazards; see Section 5.3.4.4.

## *5.3.6 Flood hazard*

### 5.3.6.1 General

Flooding of secondary urban roads can be caused by many natural hazards, e.g., excessive rain, tsunamis, the breakage of a water main, and rising sea levels (Caltrans 2018).

### 5.3.6.2 Key metric for consideration

An increase in heavy precipitation events combined with other changes in land use and land cover can increase the risk of flash flooding. Excessive precipitation can create sinkholes in urban roads or can wash away the roads. The rising sea level is another source of flooding for urban roads. For example, king tides have flooded the Embarcadero in San Francisco, including in the past 2 years (Caltrans 2018). Another source of flooding is a storm surge. Storm surges are associated with different precipitation event recurrences, such as 20- or 100-year storm events, which can cause tidal bay flooding in coastal areas.

### 5.3.6.3 Infrastructure improvements

The following countermeasures can reduce the vulnerability of urban roads to flooding:

- Provide barriers where possible.
- Improve drainage.
- Maintain roads.

#### 5.3.6.4 Cost-benefit considerations

Urban roads are sometimes shut down because of severe flooding and deep inundation. When shutdown occurs, the city and residents lose a transport function and mobility for a certain period. By improving drainage, maintaining roads, and providing barriers, at a cost of 3% of the replacement cost, damage probability can be assumed to decrease from 10% for existing roadways to 5% for improved roadways.

## 5.4 Urban (roadway) bridges

### *5.4.1 Overview*

Urban roadway bridges are a means to overcome road obstacles (e.g., rivers, railroad tracks, highways, and other roadways) to allow for an increased flow of traffic in urban areas. Urban roadway bridges also serve to connect roadways to highways. Figure 127 shows an example.

Figure 127. Roadway bridge

### *5.4.2 Summary*

The results from a literature review of urban roadway bridges are summarized in Table 46 and are presented in the "*Improvement of infrastructure resilience evaluation matrix for selected natural hazards*" table of the project final report. The following subsections of this background report provide more detailed descriptions and background information. Because roadway bridges are in cities that have a high population density of both people and buildings, they are not subject to landslide hazard; therefore, landslides are not discussed in this section.

| Hazard | Susceptible | Improvements | Damage index | | Approximate improvement cost as % of cost |
|---|---|---|---|---|---|
| | | | As-is | Improved | |
| **Earthquake** | Y | Use seismic design and detailing | 0.35 | 0.04 | 20% |
| **Liquefaction** | Y | Use steel pile foundation at abutments and bents | 0.40 | 0.10 | 30% |
| **Wind** | Y | Use connection details with a long fatigue life | 0.10 | 0.03 | 5% |
| **Flood** | Y | Add rocks at abutments and piers | 0.03 | 0.02 | 1% |
| **Landslide** | N | -- | -- | -- | -- |

Table 46. Summary of findings for urban roadway bridges

### *5.4.3 Vulnerability to natural hazards*

Urban roadway bridges have been significantly damaged in past natural hazard events. Figure 128 presents a few examples.

Earthquake (New Zealand, 2011)

Liquefaction (Japan, 1994)

Wind

Flood , scour

Figure 128. Damage to urban roadway bridges from natural hazards

### *5.4.4 Earthquake hazard*

#### 5.4.4.1 General

Urban roadway bridges are susceptible to damage from peak ground acceleration (PGA) that is caused by earthquakes. Most of the past earthquake damage to urban roadway bridges has been the result of poorly designed reinforced-concrete members and steel connections. The damage states are defined as listed in Table 47 (FEMA 2013a).

| **Damage state** | **Definition** | **Restoration, days (median)** |
|---|---|---|
| DS0 (none) | -- | -- |
| DS1 (minor) | Minor cracking and spalling to the abutment, cracks in shear keys at abutments, minor spalling and cracks at hinges, minor spalling at the column (damage requires no more than cosmetic repair) and/or minor cracking to the deck | 1 |
| DS2 (moderate) | Any column experiencing moderate (shear cracks), cracking and spalling (column structurally still sound), moderate movement of the abutment (<2”), extensive cracking and spalling of shear keys, any connection having cracked shear keys or bent bolts, keeper bar failure without unseating, rocker bearing failure and/or moderate settlement of the approach. | 3 |
| DS3 (extensive) | Any column degrading without collapse (shear failure - column structurally unsafe), significant residual movement at connections, and/or major settlement approach, vertical offset of the abutment, differential settlement at connections, shear key failure at abutments. | 75 |
| DS4 (complete) | Any column collapsing and connection losing all bearing support, which may lead to imminent deck collapse, tilting of substructure due to foundation failure. | 230 |

Table 47. Earthquake damage states for urban roadway bridges

### 5.4.4.2 Key metric for consideration

A key consideration is the amount of time that it takes to restore urban roadway bridges after an earthquake. Figure 129 presents the timeline of restoration (FEMA 2013a) for roadway bridges.

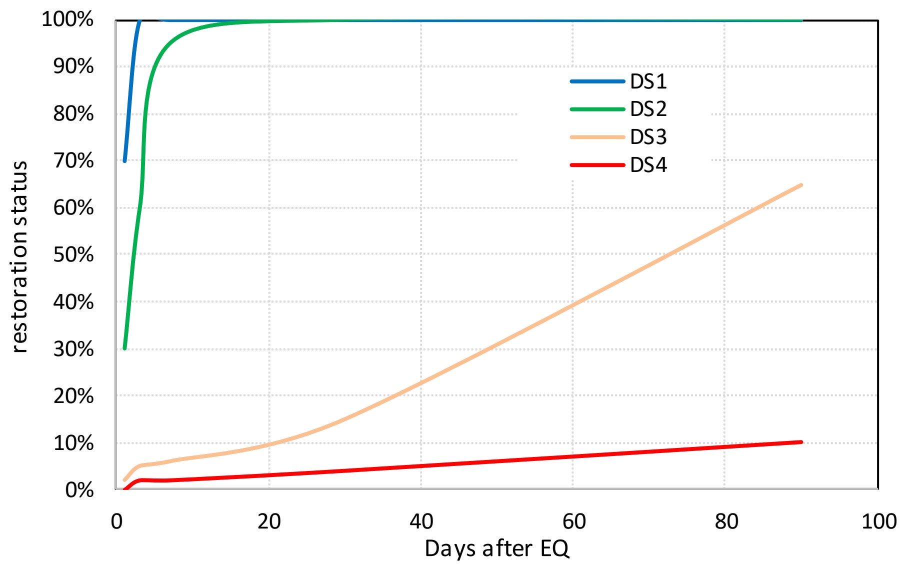



Figure 129. Restoration curve, roadway bridges, earthquake hazard

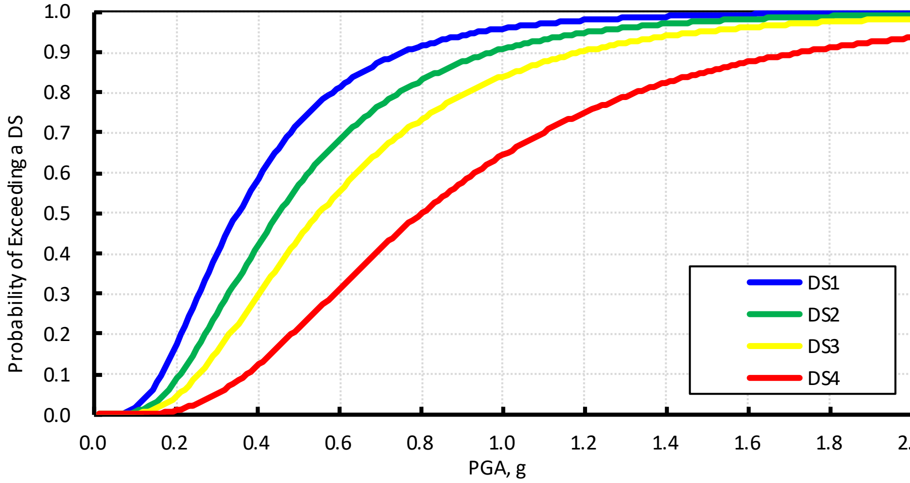
### 5.4.4.3 Infrastructure improvements

Retrofit of existing roadway bridges usually involves upgrading the bridge to DS2 or DS3 to ensure that it does not collapse if an earthquake occurs. For new bridges, earthquake performance can be improved by several methods. Examples include:

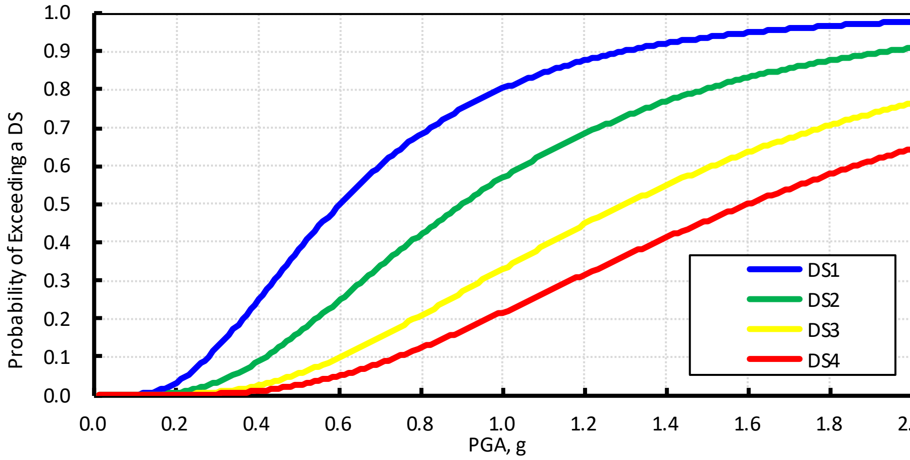
- Follow the current roadway bridge seismic design procedure that is used in California or in Japan.
- Design the column elements to act as the yielding elements (fuses) to dissipate seismic energy.
- Detail the columns to be ductile by providing closely spaced lateral reinforcement.
- During construction, perform regular inspections, including during concrete pours.
- Conduct random sampling and materials testing.

### 5.4.4.4 Cost-benefit considerations

FEMA (2013a) provides fragility functions for urban roadway bridges in the United States. In this report, single-span simply supported or single-bent (two-span) bridges (types 20 and 21 in FEMA 2013a) are used as a reference, because these types of bridges are currently the most common new bridges that are being constructed. In addition, the findings are similar for other bridges. Figure 130 presents the fragility functions for bridges with a conventional design and a seismic design, respectively. Note that for a $M_w$ 7 earthquake with a likely PGA of 0.4g, the probability of exceeding DS3 is 30% for the conventional design and is 4% for the seismic design. In other words, by using seismic components at an additional estimated cost of 20%, the probability that the bridge will be nonoperational for 2.5 months decreases from 30% to 4%. Such expedited restoration provides significant economic benefits, reduces downtime and repair costs, and helps with the community recovery.




Conventional design




Seismic design

Figure 130. Damage fragility functions for urban roadway bridges, earthquake hazard

### *5.4.5 Liquefaction hazard*

#### 5.4.5.1 General

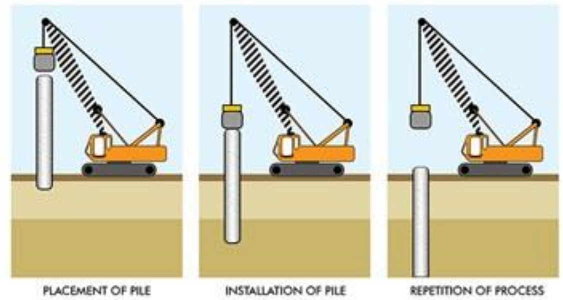
Urban roadway bridge piers that are constructed on weak soils in earthquake zones are vulnerable to damage from liquefaction.

#### 5.4.5.2 Key metric for consideration

Liquefaction can cause loss of support, resulting in roadway bridge damage or collapse of the entire bridge.

#### 5.4.5.3 Infrastructure improvements

Retrofit of existing urban roadway bridges on liquefiable soil is similar to new construction and involves enlarging the foundation and adding piles. This section therefore presents methods for new construction. For roadway bridges, liquefaction performance can be improved by using a driven-pile foundation. Another option is to use micropiles. Figure 131 shows examples.

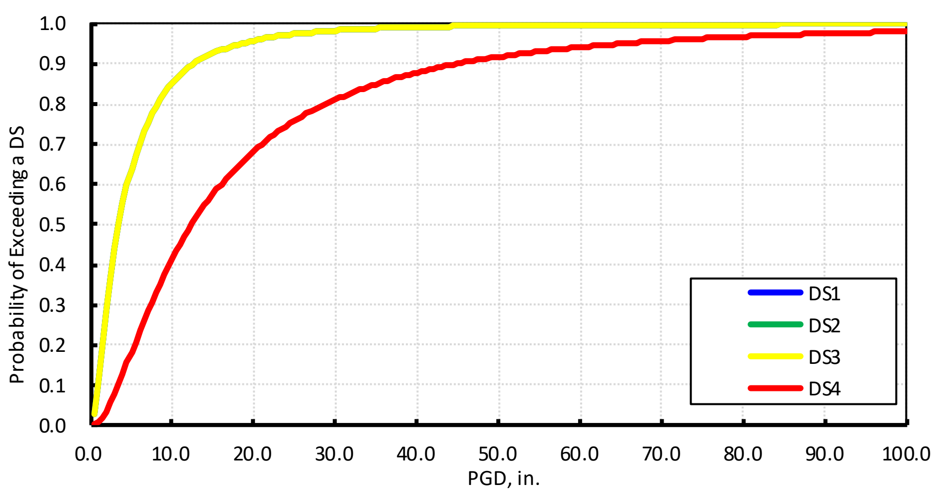



Steel H or pipe piles | Prestressed piles

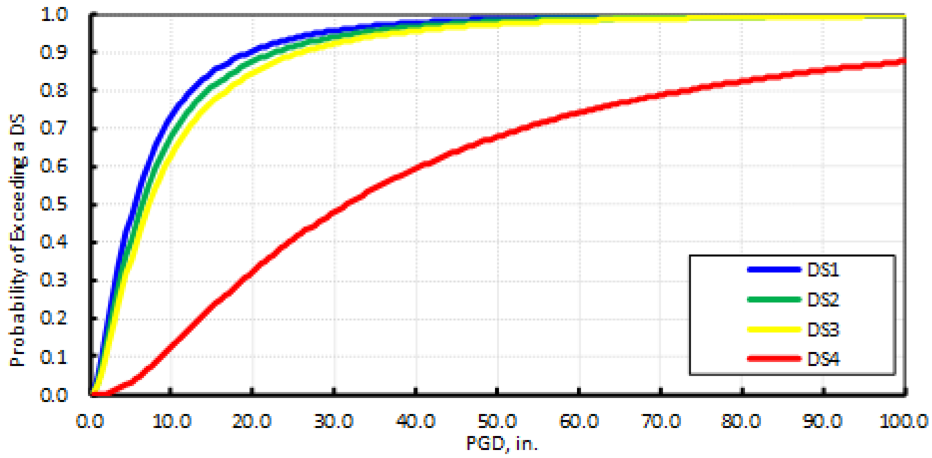
Figure 131. Examples of driven-pile foundations

### 5.4.5.4 Cost-benefit considerations

FEMA (2013a) provides fragility functions for bridges (highway or city) in the United States. In this report, that bridge data is used as a reference and is adopted for roadway bridges. Figure 132 presents the fragility functions for conventional bridge design and bridge design with liquefaction mitigation, respectively. Note that for a PGD of 10 in., the probability of exceeding DS4 is 40% for the conventional design and is 10% for the deep foundation seismic design. In other words, at an additional cost of approximately 30% for steel piling, the probability that the bridge will be nonoperational for almost 8 months decreases from 40% for the conventional design to 10% for the improved design. Such expedited restoration provides significant economic benefits, reduces downtime and repair costs, and helps with the community recovery.




Conventional design




Design to mitigate liquefaction

Figure 132. Damage fragility functions for bridges, liquefaction hazard

### *5.4.6 Wind*

#### 5.4.6.1 General

Many urban roadway bridges and highway overpasses are steel bridges. Typical steel bridges are made of built-up plate girders, which are susceptible to distortion-induced fatigue cracking where the girders (parallel to the roadway) are connected by diaphragms or by cross frames (common for roadway).

#### 5.4.6.2 Key metric for consideration

Improve the fatigue life of the bridges. The detailing of steel girder connections is the most critical element in determining the fatigue life of an urban roadway bridge

#### 5.4.6.3 Infrastructure improvements

The fatigue life of roadway bridges can be improved by several methods, including the following list and the methods in Figure 117 (Alemdar et al. 2014).

- Use pre-stressed girders.
- Ensure that the system is not fracture-critical by using at least three girders.
- Use details that have a long fatigue life.
- Perform regular maintenance and inspection of the roadway bridge. If a fatigue crack is observed, perform calculations and drill crack arrest holes.

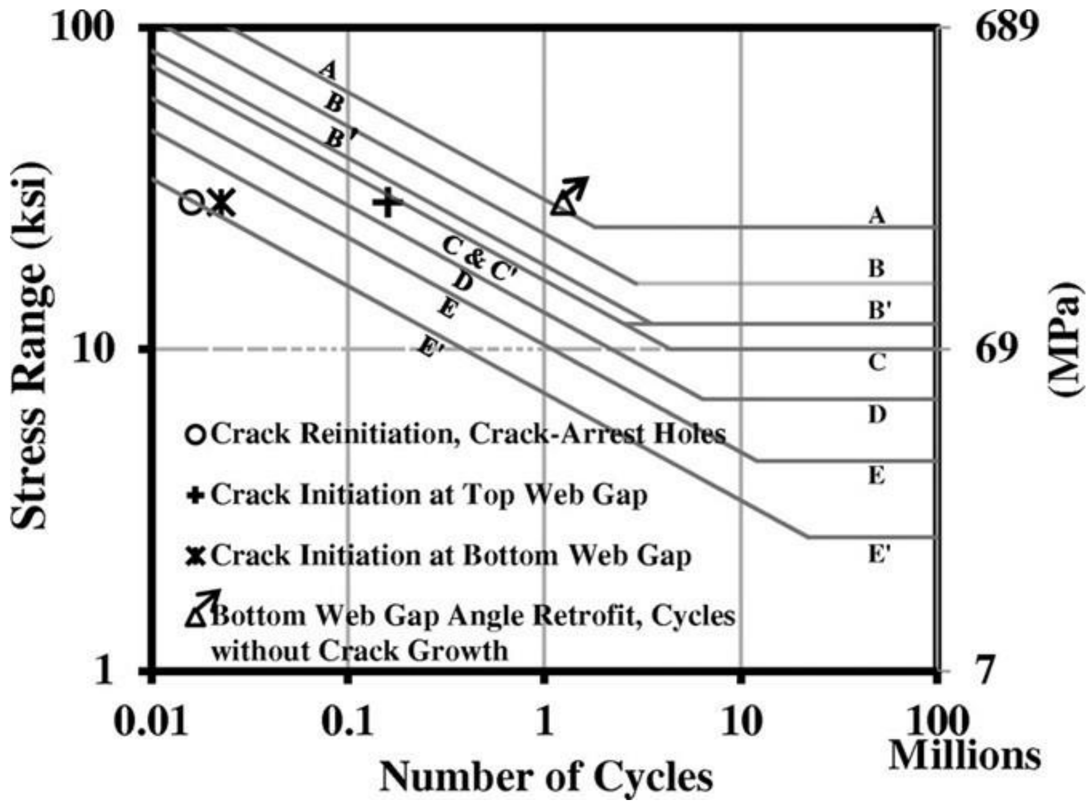



Figure 133. Examples of wind mitigation for fatigue by improved connection, urban roadway bridges

#### 5.4.6.4 Cost-benefit considerations

Connection upgrades significantly improve the fatigue performance of steel girder bridges. It is estimated that at approximately 5% of the total cost, improved connections can increase the fatigue life by a factor of 3 or more, or reduce the likelihood of damage from an estimated 10% for existing roadway bridges to 3% for improved bridges.

### *5.4.7 Flood hazard*

#### 5.4.7.1 General

Urban roadway bridges are constructed in cities and are therefore less susceptible to flooding. In most cases, the roadway bridge foundation is placed at street medians and so is less vulnerable to scour. These bridges could be damaged, however, if they are over small waterways or if roadway flash flooding occurs. Table 48 shows the vulnerability matrix that was adopted from FEMA (2013c).

| Overall vulnerability | Inundation | Scour | Hydraulic pressure & debris | Financial loss | Loss of operation |
|---|---|---|---|---|---|
| High | Low | Medium/low | Low | Medium | Medium |

Table 48. Flood vulnerability for roadway bridges

#### 5.4.7.2 Key metric for consideration

Figure 134 presents the status of damage (FEMA 2013c) for urban bridges, adopted from single-span highway bridges. Note that the potential for scour and damage increases significantly for larger events.

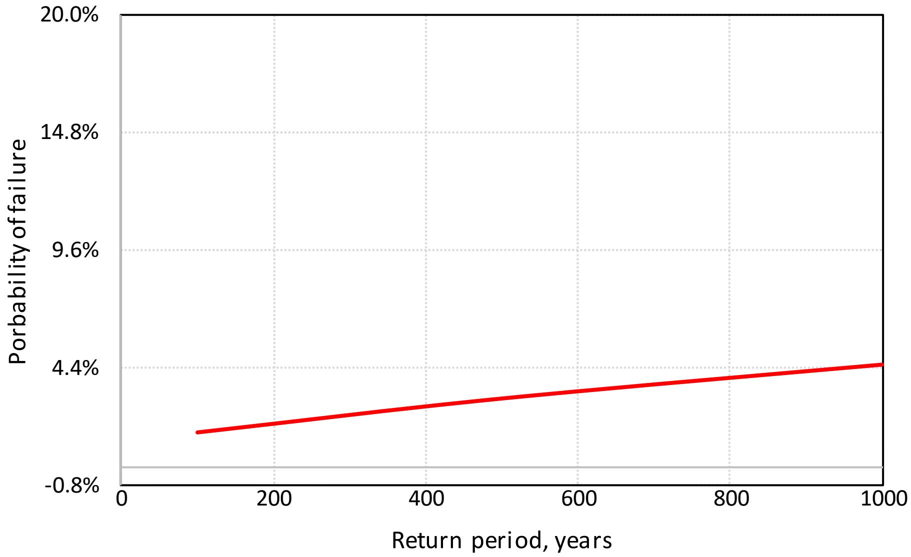


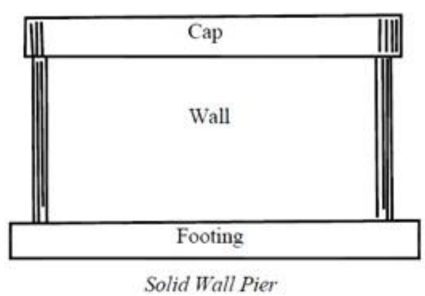

Figure 134. Percentage of damage as a function of inundation depth

### 5.4.7.3 Infrastructure improvements

Following are examples of flood performance improvements for urban roadway bridges. Figure 135 also shows examples:

- Use pier walls for multi-bent bridges to reduce undermining of column piers.
- Excavate and backfill with approximately 1 m of rock around the column footing.
- Remove any debris and pieces of wood that are against the piers.
- Conduct regular maintenance and inspection of the bridge, the abutment, and footings.

Rockfill




Pier wall

Figure 135. Examples of scour mitigation for urban roadway bridge piers

### 5.4.7.4 Cost-benefit considerations

The cost of adding rock to an abutment or around the column footing of urban roadway bridges is estimated at 1% of the replacement cost, and the performance is assumed to improve from a damage probability of 3% for existing roadway bridges to 2% for improved bridges.

## 5.5 Tertiary roads

### *5.5.1 Overview*

Tertiary or local roads are the transportation structures that carry traffic from villages and isolated dwellings to other components of the transportation network. Figure 136 shows examples.

Urban gravel road

Rural dirt road

Figure 136. Tertiary roads

### *5.5.2 Summary*

The results from a literature review of tertiary roads are summarized in Table 49 and are presented in the "*Improvement of infrastructure resilience evaluation matrix for selected natural hazards*" table of the project final report. The following subsections of this background report provide more detailed descriptions and background information. On-grade roadways are not susceptible to damage from direct wind; thus wind hazard is not discussed in this report. Hurricane-induced flooding is not mentioned specifically but is encompassed in the flood hazard section (Section 5.5.6).

| Hazard | Susceptible | Improvements | Damage index | | Approximate improvement cost as % of cost |
|---|---|---|---|---|---|
| | | | As-is | Improved | |
| **Earthquake** | Y | Compaction of the underlying material, earthquake-resistant foundations | 0.10 | 0.05 | 10% |
| **Liquefaction** | Y | Soil improvement | 0.10 | 0.05 | 5% |
| **Wind** | N*** | -- | -- | -- | -- |
| **Flood** | Y | Barriers and drainage | 0.10 | 0.05 | 3% |
| **Landslide** | Y | Geometric reconfiguration, structural and hydrological solutions, retaining wall, stabilize slope, shotcrete, soil nails | 0.20 | 0.02 | 5% |

Table 49. Summary of findings for tertiary roads

### *5.5.3 Vulnerability to natural hazards*

Tertiary roads have been significantly damaged in past natural hazard events. Figure 137 presents examples.

*** Wind hazards such as tornadoes could result in loss of part of a dirt road surface, but this effect is not considered to be critical.

Earthquake (Napa, California, USA, 2014)

Liquefaction (New Zealand, 2011)

Landslide

Flood (road washed away, Colorado, USA, 2013)

Figure 137. Damage to tertiary roads from natural hazards

### *5.5.4 Earthquake hazard*

#### 5.5.4.1 General

Tertiary roads are susceptible to damage from permanent ground displacement (PGD) that is caused by earthquake shaking. Most of the past earthquake damage to tertiary roads has been the result of underlying material that is poorly compacted. The damage states are similar to those of urban roads and are listed in Table 50 (FEMA 2013a). Note that both complete damage and extensive damage are combined into a single damage state, DS3, for tertiary roads.

| Damage state | Definition | Restoration, days (median) |
|---|---|---|
| DS1 | Defined by slight settlement (few inches) and/or offset of the ground. | 1 |
| DS2 | Defined by moderate settlement (several inches) and/or offset of the ground. | 3 |
| DS3 | Defined by major settlement of the ground (few feet). | 30 |

Table 50. Earthquake damage states for tertiary roads

#### 5.5.4.2 Key metric for consideration

A key consideration is the amount of time that it takes to restore tertiary roads after an earthquake. Figure 138 shows the timeline of restoration for tertiary roads. Note that tertiary roads need less restoration time than urban roads do, because tertiary roads have less demanding roughness tolerances and fewer layers/components. In many cases, depending on the number of people that tertiary roads serve, these roads have the lowest priority.

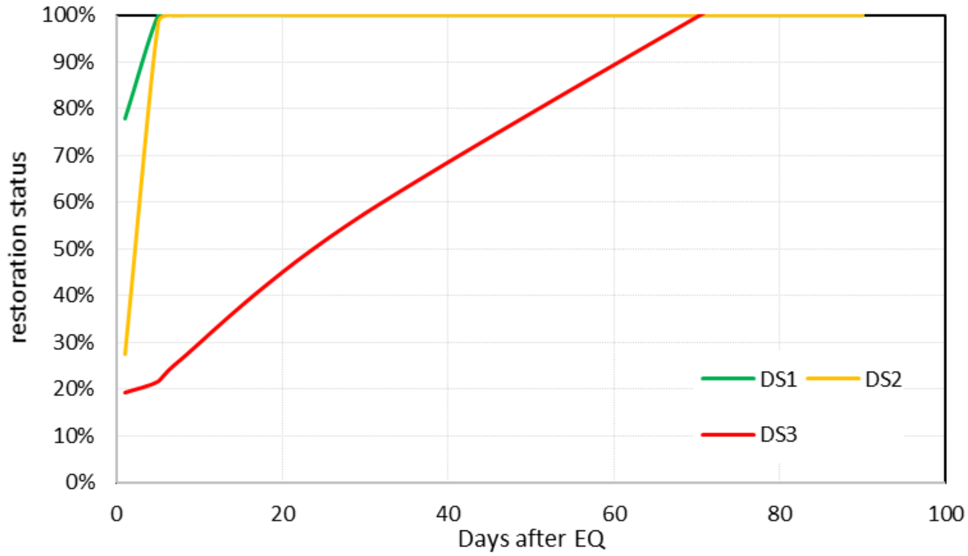



Figure 138. Restoration curve, tertiary roads, earthquake hazard

### 5.5.4.3 Infrastructure improvements

Retrofit of existing tertiary roads is relatively inexpensive. For new construction, earthquake performance can be improved by several methods to reduce risk from DS3 to DS2. Examples include (FEMA 2006):

- Properly compact the underlying material.
- Use earthquake-resistant foundations.

### 5.5.4.4 Cost-benefit considerations

Figure 139 presents the fragility functions for tertiary roads for the United States. The probability of failure for tertiary roads is lower than for urban roads, because tertiary roads have less demanding roughness tolerances and fewer layers/components. By the improvements, the expected PGD is assumed to reduce from 12 in to 8in which means the factor is 2 (i.e., DS3 damage probability comes from 10% to 5%The repair and restoration time difference for the two scenarios is 70 days versus 5 days, respectively. Therefore, by introducing seismic improvements (e.g., removal of old pavement, compaction of the underlying material, and installation of new payment) at 10% of the replacement cost, the restoration time can be reduced from 70 days to 5 days.

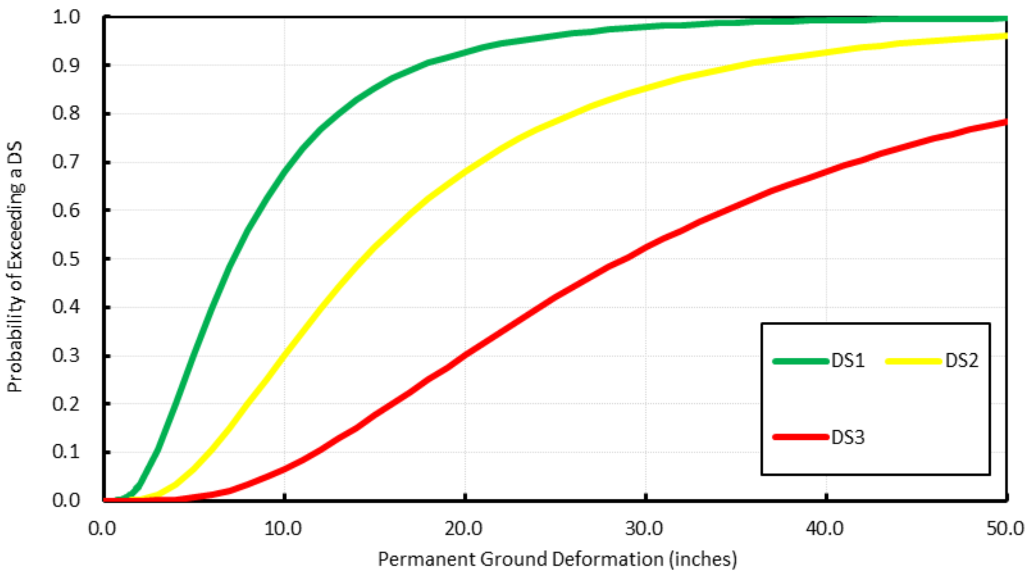



Figure 139. Damage fragility functions for tertiary roads

### *5.5.5 Liquefaction hazard*

#### 5.5.5.1 General

Tertiary roads that are constructed on weak soils in earthquake zones are vulnerable to damage from liquefaction.

#### 5.5.5.2 Key metric for consideration

Liquefaction can result in damage and loss of operation of tertiary roads due to large ground deformation.

#### 5.5.5.3 Infrastructure improvements

Retrofit of existing tertiary roadways on liquefiable soil is not cost-effective. For new tertiary roads, liquefaction performance can be increased by several methods that improve the quality of the underlying soil. Figure 140 shows some typical examples.

Vibro compation of a gravel road


Dynamic compaction of a gravel road

Figure 140. Examples of geotechnical liquefaction mitigation for tertiary roads

#### 5.5.5.4 Cost-benefit considerations

For tertiary roads, the benefits of liquefaction improvement methods are similar to the benefits of improvements against earthquake hazards see Section 5.5.4.4.

### *5.5.6 Flood hazard*

#### 5.5.6.1 General

Flooding of tertiary roads can be caused by many natural hazards, e.g., excessive rain, tsunamis, the breakage of a water main, and rising sea levels (Caltrans 2018).

#### 5.5.6.2 Key metric for consideration

Some unpaved or gravel roads are in mountainous areas and are subject to flooding from downpour and nearby streams. Typically, a 100-year flood or storm event is used as a metric.

#### 5.5.6.3 Infrastructure improvements

The following countermeasures can reduce the vulnerability of tertiary roads to flooding:

- Pave with gravel if feasible.
- Slope embankments to allow water runoff to the side and away from the road.
- Provide road barriers at susceptible segments.

#### 5.5.6.4 Cost-benefit considerations

Tertiary unpaved road surfaces could wash away during flooding. The addition of gravel, paving, or providing drainage slopes is estimated to cost approximately 3% and can reduce damage probability, from 10% for existing tertiary roads to 5% for improved roads.

### *5.5.7 Landslide*

#### 5.5.7.1 General

Tertiary roads in mountainous regions are susceptible to damage and to service interruptions that can last months because of landslides.

#### 5.5.7.2 Key metric for consideration

Depending on the extent of the landslide, the cost of repair can vary dramatically from very inexpensive (small cleanup) to complete closure of the road and rerouting of traffic while an alternative road is built.

#### 5.5.7.3 Infrastructure improvements

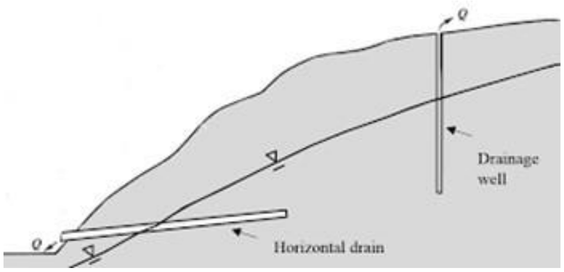
For tertiary roads, resistance to landslides can be improved by several methods. Examples are outlined in the following bullet list and are shown in Figure 141:

- Geometric (slope) reconfiguration and removal of unstable soil
- Mechanical/structural solutions:
    - Install cantilever retaining walls
    - Install drainpipes
- Hydrological solutions;
    - Add drainage to reduce water pressure
    - Prevent water from entering the hillside by diversion
- Erosion control:

- Add steel netting
- Add geomats
- Add coconut fiber mesh
- Build a vegetated wall system

Retaining walls

Vegetated wall system




Drainpipes

Drainpipes (installation example)

Erosion control by hydroseeding

Slope controlling vegetation (installation example)

Figure 141. Examples of landslide mitigation techniques for tertiary roads

### 5.5.7.4 Cost-benefit considerations

The use of cantilever retaining walls is one of the most common methods to mitigate landslides. The additional cost of the retaining walls is estimated to be 5% of the total cost, based on the assumption that only a limited section of the tertiary road requires protection. Landslides can cause tertiary roads to be shut down for months at a time. Therefore, in this report, it is assumed that mitigation will reduce the risk of loss of operation and damage by a factor of 10: from 20% for existing tertiary roads to 2% for improved roads.

## 5.6 Wooden bridges

### *5.6.1 Overview*

Wooden bridges can generally be found in rural areas and are usually categorized as a minor arterial, collector, or local road. Figure 142 shows examples. Most wooden bridges were built a long time ago and are based on non-modern bridge design codes, and some have been registered as part of a historic bridge inventory. Therefore, wooden bridges are a relatively vulnerable component of the roadway network, and they have different weak elements and points depending on the natural disaster that occurs.


Figure 142. Wooden bridges

### *5.6.2 Summary*

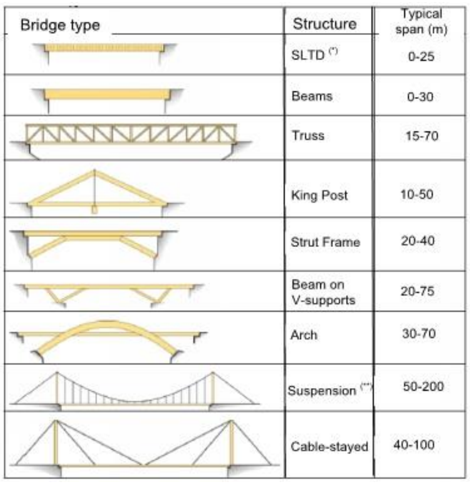
The results from a literature review of wooden bridges are summarized in Table 51 and are presented in the "*Improvement of infrastructure resilience evaluation matrix for selected natural hazards*" table of the project final report. In Table 51, the improvement cost is simply expressed as the ratio of the improvement cost to the component replacement cost, and the resiliency index is estimated as a probability of exceeding severe damage (i.e., more than severe) when the hazard threshold intensity occurs. The following subsections of this background report provide more detailed descriptions and background information.

| Hazard | Susceptible | Improvements | Resiliency index | | Approximate improvement cost as % of cost |
|---|---|---|---|---|---|
| | | | As-is | Improved | |
| **Earthquake** | Y | Wood element and connection strengthening | 0.35 | 0.03 | 20% |
| **Liquefaction** | Y | Foundation retrofit (piles and footings) | 0.44 | 0.13 | 30% |
| **Wind** | Y | Connection retrofit/replacement for fatigue capacity | 0.15 | 0.05 | 10% |
| **Flood** | Y | Scour mitigation by ground improvements | 0.06 | 0.02 | 3% |
| **Landslide** | Y | Soil and ground improvements | 0.63 | 0.25 | 25% |

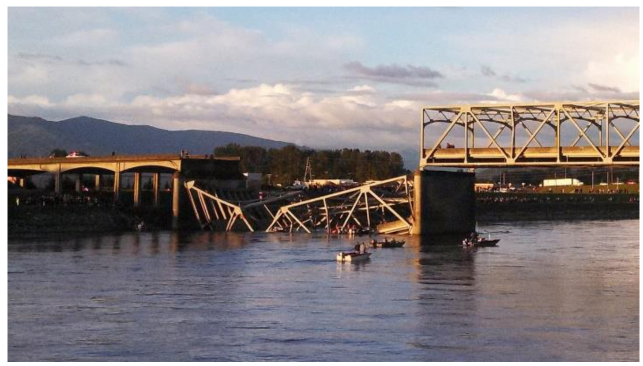
Table 51. Summary of findings for wooden bridges

### *5.6.3 Vulnerability to natural hazards*

The first illustration in Figure 143 shows the typical structural types of wooden bridges. In this report, several structural systems are applied based on the bridge span and design loads, then damage scenarios from natural disasters are related to the structural system and to the disaster load. Wooden bridges have been significantly damaged in past natural hazard events, as the photographs in Figure 143 exemplify. For earthquake motions, the critical elements of a wooden bridge are the wood structural members and connections; those elements in similar types of bridges have been seriously damaged in past earthquakes (e.g., truss failure). In earthquake-induced liquefaction, the bridge foundation and substructure are especially vulnerable to ground movement and to failure, so those elements are considered to be critical. In a wind hazard, the connections of a bridge structure are assumed to be more critical than the bridge structural element itself. Bridge connections have undergone fatigue for a longer time, and wind load is typically a cyclic load that deteriorates fatigue strength. In a flood disaster, scour at the riverbed of a bridge pier is known to be the most critical damage scenario, because movement or settlement of a bridge pier causes severe damage to the bridge itself. For a landslide, the large movement of surrounding soil affects the entire bridge, through the foundation and the piers; therefore, the unstable soil is considered to be the critical component.



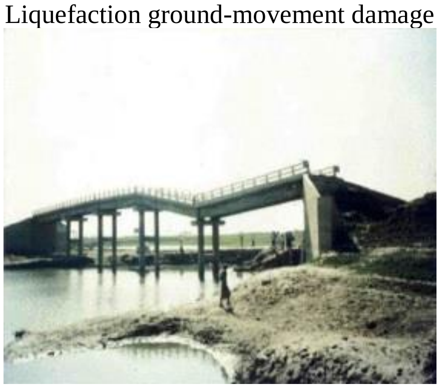

Wooden bridge structural types



Earthquake truss failure


Liquefaction ground-movement damage

Long-period and multidirectional wind damage



Flood scour damage

Landslide damage

Figure 143. Structural types and disaster damage for bridges that are similar to wooden bridges

### *5.6.4 Earthquake hazard*

#### 5.6.4.1 General

The level of damage to a wooden bridge from an earthquake is greatly affected by the seismic acceleration (peak ground acceleration [PGA], spectral acceleration [Sa], etc.) that earthquake motion causes. Most of the past seismic damage to wooden bridges has been the result of overstressed elements and poor member connections. The general damage states of bridges from earthquakes are defined as listed in Table 52 (FEMA 2013a).

| **Damage state** | **Definition** |
|---|---|
| DS0 (none) | -- |
| DS1 (minor) | Minor cracking and spalling to the abutment, cracks in shear keys at abutments, minor spalling and cracks at hinges, minor spalling at the column (damage requires no more than cosmetic repair) and/or minor cracking to the deck. |
| DS2 (moderate) | Any column experiencing moderate (shear cracks) cracking and spalling (column structurally still sound), moderate movement of the abutment (<2"), extensive cracking and spalling of shear keys, any connection having cracked shear keys or bent bolts, keeper bar failure without unseating, rocker bearing failure and/or moderate settlement of the approach. |
| DS3 (extensive) | Any column degrading without collapse (shear failure - column structurally unsafe), significant residual movement at connections, and/or major settlement approach, vertical offset of the abutment, differential settlement at connections, shear key failure at abutments. |
| DS4 (complete) | Any column collapsing and connection losing all bearing support, which may lead to imminent deck collapse, tilting of substructure due to foundation failure. |

Table 52. Earthquake damage states for bridges

#### 5.6.4.2 Key metric for consideration

The intensity threshold that causes damage to a wooden bridge is assumed to be a seismic acceleration of 0.4g. A damage state of more than DS3 (i.e., more than extensive/severe damage) is adopted to estimate the resiliency index, such as damage probability. In addition, Figure 144 (FEMA 2013a) shows a timeline of restoration and recovery for typical roadway bridges after an earthquake.

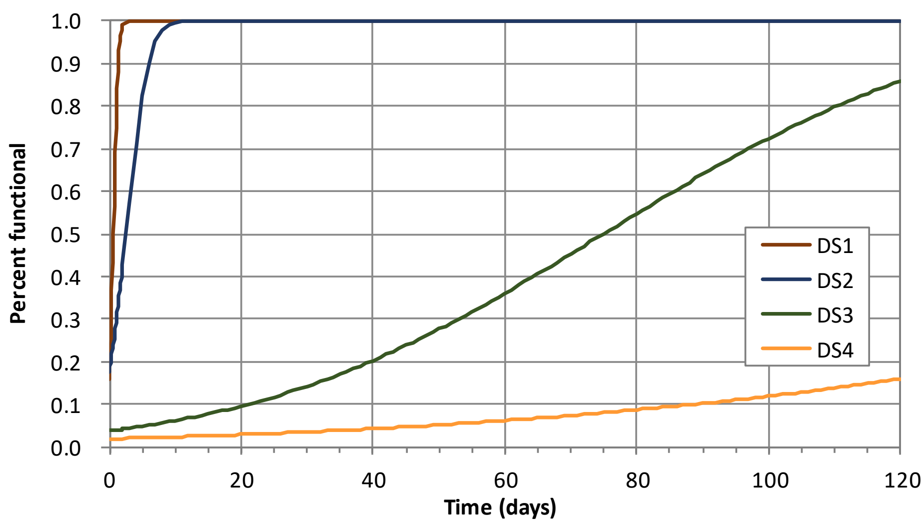



Figure 144. Restoration curve for roadway bridges, earthquake hazard

### 5.6.4.3 Infrastructure improvements

Retrofit of existing wooden bridges usually involves upgrading the bridge to DS2 or DS3 to ensure that the bridge does not collapse if an earthquake occurs. For new bridges, earthquake performance can be improved by several methods. For the wood elements and connections, adding new supportive members and connection parts improves the element strength and connection capacity, and is an effective engineering improvement for exiting wooden bridges (FHWA 2005). Quality assurance (QA) typically consists of materials testing and construction inspection. Based on engineering judgment, it is assumed that the cost of standard QA is about 10% of the project budget, and the higher-level QA cost is approximately 15% of the total budget. Therefore, for better QA of retrofit work, the higher-level QA approach is considered in this report, as shown in the last bullet, and the standard QA is assumed to have been applied to existing bridges.

Improvements to increase the earthquake resistance of wooden bridges include:

- Strengthen the wood elements and connections by adding new supportive members and connection parts; see Figure 145.
- Follow the current bridge seismic design procedure that is used in California or in Japan.
- Ensure that the foundation remains elastic during earthquakes.
- Design in-span hinges with sufficient width to prevent unseating of the bridge, even in very large earthquakes.
- Perform routine and regular maintenance for all components and fix any observed problems.
- Conduct the higher-level QA protocol (e.g., daily testing and continual and detailed field inspection).

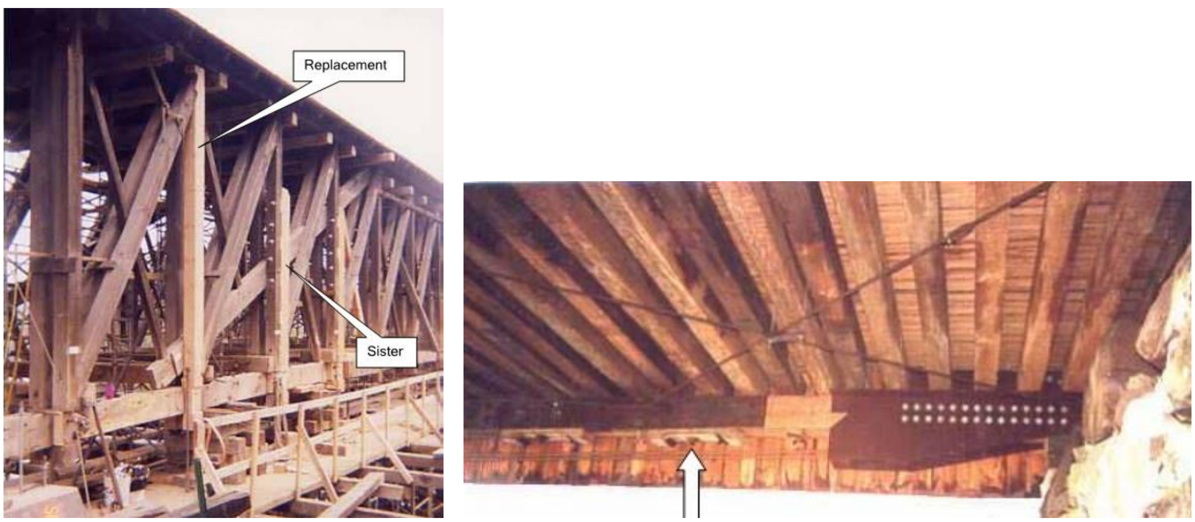
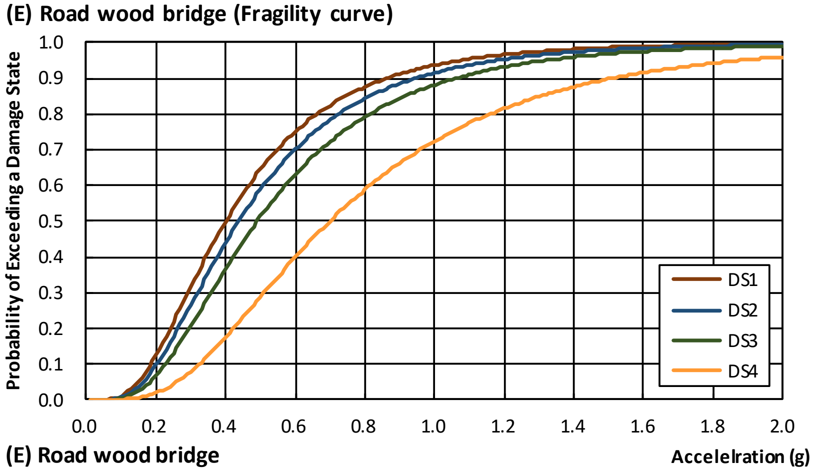



Figure 145. Structural element and connection improvements for wooden bridges

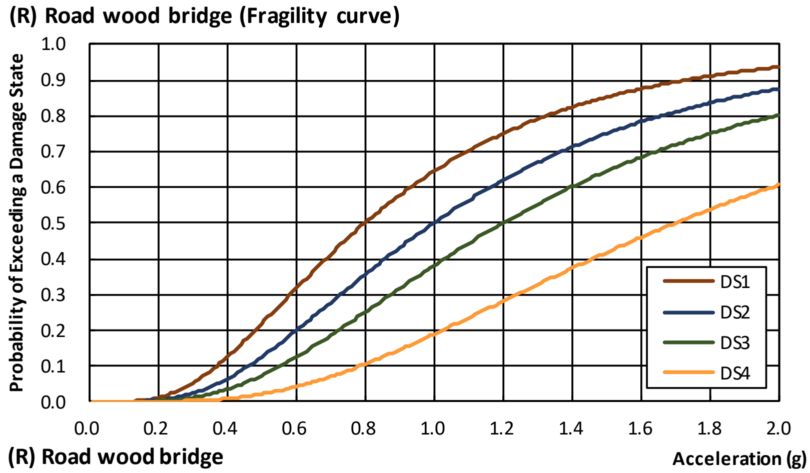
### 5.6.4.4 Cost-benefit considerations

FEMA (2013a) provides seismic fragility functions for roadway bridges in the United States. Two sets of fragility curves are provided for wooden bridges: one for new construction that is based on seismic design and one for a conventional wooden bridge that has been adjusted based on the fragility curve for the seismic design. The fragility parameters of these functions are then used to estimate the damage probability (i.e., resiliency index) for wooden bridges from seismic motion. Figure 146 shows the fragility functions for the conventional design and for the seismic design, respectively. For an earthquake with an acceleration of 0.4g, the probability of exceeding DS3 is 35% for the conventional design and is 3% for the improved wooden bridge.

The cost to improve the seismic capacity of a wooden bridge is estimated to be approximately 20% of the replacement cost for bridge itself. The total improvement cost, including the higher-level QA, can then be estimated as 20% of the total cost of new wooden bridge construction. Because the damage probability is reduced by implementation of the improvement measures, the damage level of a wooden bridge will likely be lower during any earthquake. This improved performance also means a shorter restoration time, which provides significant transportation benefits and helps with the community recovery.




Conventional design




Seismic design

Figure 146. Seismic fragility functions for wooden bridges, earthquake hazard

### *5.6.5 Liquefaction hazard*

#### 5.6.5.1 General

Wooden bridges are damaged by permanent ground deformation (PGD) such as horizontally forced displacement and lateral spreading of foundation soil caused by earthquake-induced liquefaction (FEMA 2013a). Wooden bridges that are on liquefiable and weak soils in high-seismicity zones are very vulnerable to such liquefaction damage.

#### 5.6.5.2 Key metric for consideration

The intensity threshold that damages wooden bridges is assumed to be the same as for other bridges (PGD of 10 in.), and damage states of more than extensive/severe damage (i.e., DS4) are adopted to represent the resiliency index (i.e., damage probability). Liquefaction can physically damage wooden bridges and cause operational loss of bridge transportation. Because the failure affects the entire bridge structure, the cost of geotechnical repair and restoration of operation is relatively high.

### 5.6.5.3 Infrastructure improvements

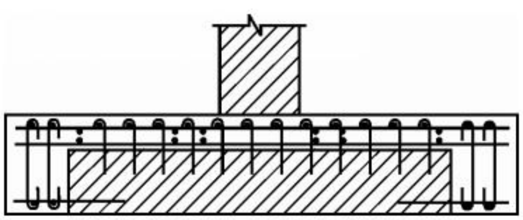
For a wooden bridge structure, liquefaction resistance can be improved by several methods, including the measures in the following bullet list. For the bridge foundation, which is a critical element, installation of additional piles and enlargement of footings, as proposed in the first bullet, strengthen the foundation against earthquake-induced liquefaction. These countermeasures are also an effective engineering improvement for existing wooden bridges (FHWA 2006). These methods improve the lateral resistance of the foundation against large soil movements. As stated in Section 5.6.4.3, it is assumed that the standard QA cost is about 10% of the project budget, and the higher-level QA cost is approximately 15% of the total budget. Therefore, for better QA of retrofit work, the higher-level QA approach is considered in this report, as proposed in the last bullet, and the standard QA is assumed to have been applied to the existing bridge construction.

Improvements to increase the liquefaction resistance of wooden bridges include:

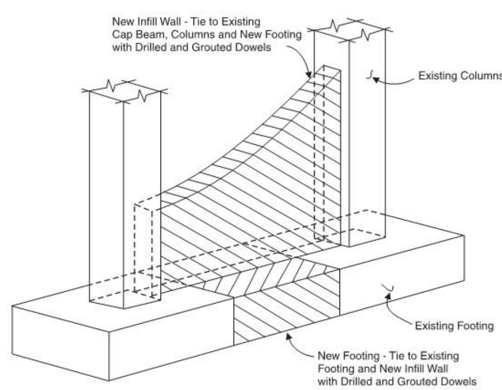
- Install additional piles and enlarge the existing footing; see Figure 147.
- Apply soil improvements and densification by several measures (e.g., cement mixing and soil compaction).
- Install drainage and drainpipes to reduce the amount of water in the soil.
- Conduct the higher-level QA protocol (e.g., daily testing and continual and detailed field inspection).

Steel pile addition


Concrete pile addition



Footing enlargement

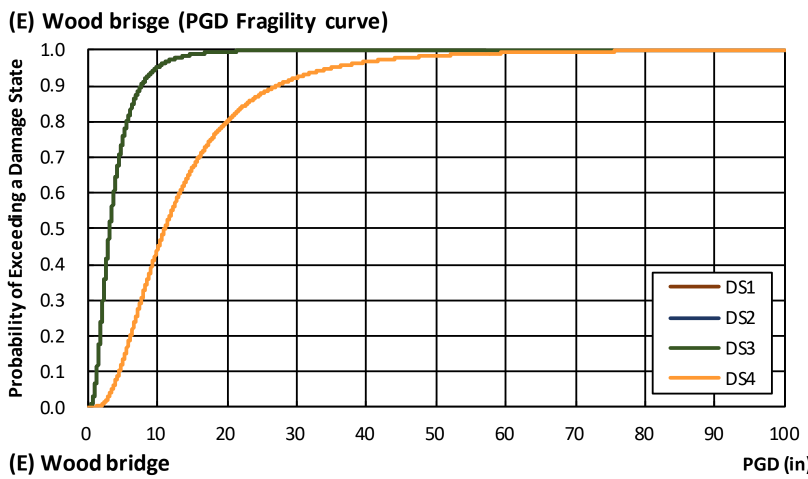



Tiying of multiple footings

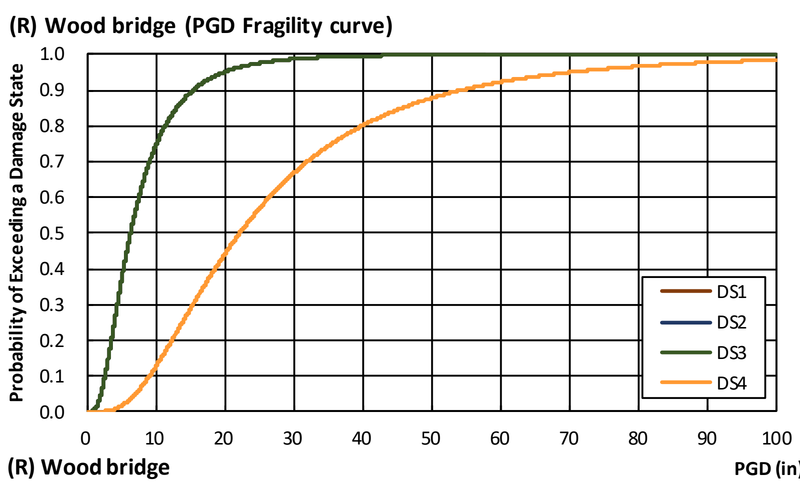
Figure 147. Examples of foundation improvements for wooden bridges

### 5.6.5.4 Cost-benefit considerations

FEMA (2013a) provides seismic fragility functions for roadway bridges that are constructed in the United States. For wooden bridges, a fragility curve that is based on PGD is provided, and the fragility curves for the conventional wooden bridge and for the retrofitted wooden bridge are developed based on that PGD fragility curve. The two sets of fragility functions, as shown in Figure 148, are then applied, resulting in the fragility functions for the conventional wooden bridge and the improved wooden bridge, respectively. The fragility parameters of these functions are used to identify the damage probability (i.e., resiliency index) for wooden bridges from seismically induced liquefaction. For earthquake-induced liquefaction with a PGD of 10 in., the probability of exceeding extensive damage (i.e., DS4) is 44% for the conventional bridge and is 13% for the improved wooden bridge. The cost of these liquefaction-resistance improvements (i.e., added piles and enlarged footings) is estimated to be approximately 30% of the replacement cost for wooden bridge itself. The total improvement cost, including the higher-level QA, can then be estimated at 30% of the total cost of wooden bridge construction. The damage probability is significantly reduced by implementation of the improvement measures, and the damage level of the wooden bridge will likely be lower in any earthquake-induced liquefaction event.




Conventional design




Seismic design

Figure 148. Damage fragility functions for wooden bridges, liquefaction hazard

### *5.6.6 Wind*

#### 5.6.6.1 General

Most wooden bridges use mechanically fastened joints (e.g., steel bolt connections) to connect the wood elements. These fastener connections are susceptible to fatigue failure due to repeating loads over an extended period, such as wind forces. Multidirectional wind loading can cause stress reversal at connections in wooden bridges. Thus, if these connections are not adequately designed, high-cycle, low-amplitude fatigue can cause them to fail from along-wind loading at stresses that are much lower than their nominal capacity.

#### 5.6.6.2 Key metric for consideration

The detailing and material of fastener connections is the most critical factor to determine the fatigue life of a wooden bridge.

#### 5.6.6.3 Infrastructure improvements

For wooden bridges, the fatigue strength of connections (i.e., the critical factor for wind resistance) can be improved by several methods, as outlined in the following bullet list. The strengthening or replacement of connection parts, as itemized in the first bullet, improves the fatigue strength and, consequently, the wind capacity. This retrofit lengthens the fatigue life of connections and keeps the wooden bridge robust enough to resist the current design wind load. As in other sections of this report, it is similarly assumed, based on engineering judgment, that the standard-QA cost is about 10% of the project budget, and the higher-level QA cost is approximately 15% of the total budget. Therefore, for better QA of retrofit work, the higher-level QA approach is considered, as listed in the last bullet, and the standard QA is assumed to have been applied to the existing connection parts.

Improvements to increase the wind resistance of wooden bridges include:

- Strengthen or replace the connections of wood elements; see Figure 149.
- Use details that have a long fatigue life for element connections.
- Use a longer design life (i.e., apply a realistic bridge life that is longer than 50 or 75 years).
- Use a conservative design because bridge loading tends to increase over time as the vehicle payloads increase.
- Conduct the higher-level QA protocol (e.g., daily testing and continual and detailed field inspection).

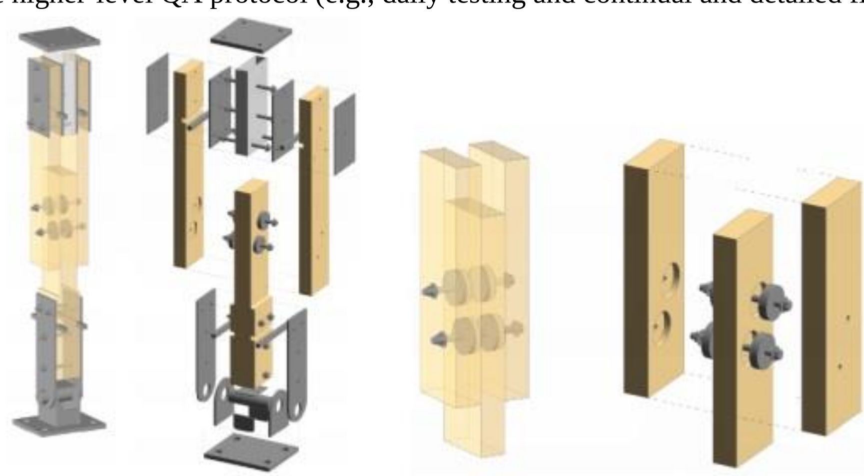


Figure 149. Conceptual figure of wood connection strengthening (Ogorzalek et al. 2017)

#### 5.6.6.4 Cost-benefit considerations

As Figure 150 shows, connection details that have a long ("infinite") fatigue life contribute to the design of a bridge that has a much larger capacity. Therefore, the connection upgrades (e.g., improvement from Category E to Category B in Figure 150) significantly improve the fatigue performance of wooden bridges. The improvement cost is estimated at approximately 10% of the replacement cost of wooden bridge itself. Connections can be improved to increase the fatigue life by a factor of 3 or more, reducing the likelihood of damage from an estimated 15% for existing wooden bridges to 5% for retrofitted bridges. Therefore, the total improvement cost, including the higher-level QA, can be estimated at 10% of the total cost of wooden bridge construction. Because the damage probability during a wind disaster is reduced by implementation of the improvement measure, the recovery process for a wooden bridge will likely be faster, as will the restart of roadway operation.

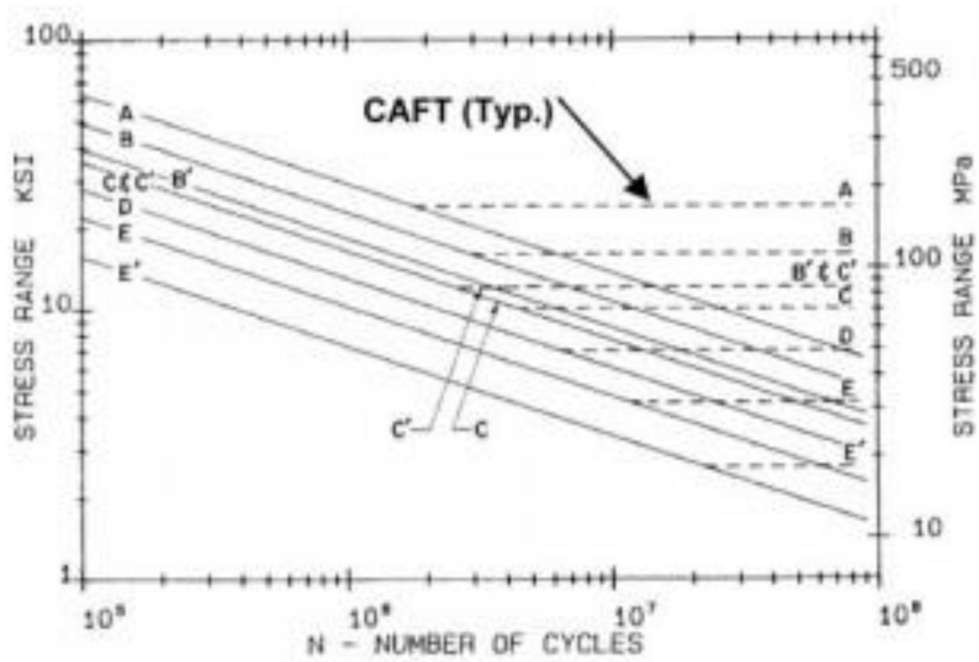



Figure 150. Fatigue capacity according to connection category (AASHTO 2017)

### *5.6.7 Flood hazard*

#### 5.6.7.1 General

It is assumed that the wooden bridge superstructure is constructed high enough that there is little chance of overtopping from floodwater. Bridge piers, however, are susceptible to damage from flooding. Floodwater removes the sediment from around piers, causing serious scour and inducing bridge failure (FEMA 2013c).

#### 5.6.7.2 Key metric for consideration

The probability of failure due to scour for roadway bridges (FEMA 2013c) is adjusted for wooden bridges by reducing the resistance to scour because of the relatively weak sediment at riverbed due to longer period, see Figure 151. Note that the potential for scour and bridge failure increases exponentially for larger events.

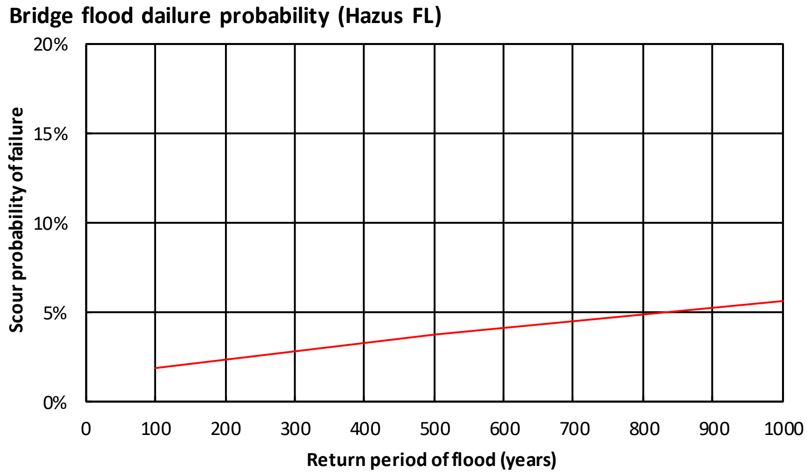



Figure 151. Percentage of damage as a function of flood return period

#### 5.6.7.3 Infrastructure improvements

For wooden bridges, the capability to resist flood scour can be upgraded by several methods, as outlined in the following bullet list. The riverbed and sediment improvement around piers and abutments, as proposed in the first two bullets, reduces the probability of scour damage and is a recommended improvement method for existing wooden bridges. As mentioned in other hazard retrofits, it is also assumed that the standard-QA cost is about 10% of the project budget, and the higher-level QA cost is approximately 15% of the total budget. Therefore, for better QA of improvement work, the higher-level QA approach is considered in this report, as listed at the last bullet, and the standard QA is assumed to have been applied to the existing wooden bridges.

Improvements to increase flood resistance for wooden bridges include:

- Use riprap (RSP, or rock slope protection) to protect the bridge piers; see Figure 152 (FHWA 2009).
- Excavate and backfill with approximately 1 m of rock around the column footings and abutments; see Figure 152.
- Conduct a hydrological study and accurately estimate the flow rate, including all tributaries.
- Align the bridge piers to mitigate the scour; for example, use oval instead of rectangular foundations.
- Use large foundations to ensure bridge stability if scour occurs.
- Conduct the higher-level QA protocol (e.g., daily testing and continual and detailed field inspection).


Figure 152. Example of scour mitigation for bridge piers and abutments for wooden bridges

#### 5.6.7.4 Cost-benefit considerations

For wooden bridge piers, it is assumed that the additional cost of the proposed scour mitigation is 3% of the replacement cost of bridge itself. Therefore, the total improvement cost, including the higher-level QA, can be estimated as 3% of the total cost of wooden bridge construction. It is also assumed that proper mitigation is similar to reducing the probability of failure from that of the 1,000-year flood to that of the 100-year flood; therefore, from 6% for existing wooden bridges to 2% for improved bridges. The damage probability is reduced by implementation of the improvement measure, so the recovery process for wooden bridges and the restoration of roadway operation will both be faster.

### *5.6.8 Landslide*

#### 5.6.8.1 General

Regardless of their structural type, bridges that are in landslide-susceptible areas are likely to be damaged from large permanent ground deformation (PGD) such as horizontally forced displacement that is caused by landslides (FEMA 2013a). If geotechnical aspects and bridges in those areas have not been designed by considering large displacements and huge soil loading, they are particularly vulnerable to such landslide hazards.

#### 5.6.8.2 Key metric for consideration

As assumed in previous sections of this report, the intensity threshold that severely damages a wooden bridge is considered to be a PGD of 14 in. from a landslide (FEMA 2013a), and the complete damage state (DS4) is adopted to represent the resiliency index, evaluated by damage probability. Landslides can physically damage a bridge and roadway network and cause functional loss of both. Because the failure of the bridge and the soil are extensive and the repair work is very difficult and takes long time, the cost of geotechnical restoration and bridge structural repair could be high.

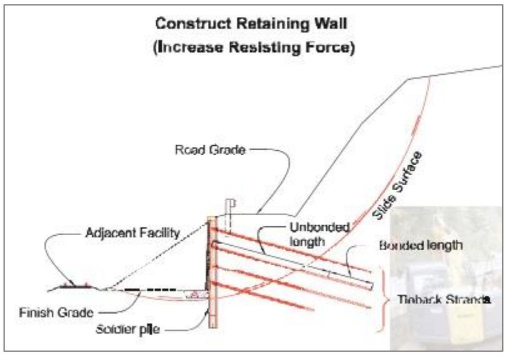
#### 5.6.8.3 Infrastructure improvements

To resist the large permanent ground movements that occur during a landslide, this report adopts the improvement of soil as a countermeasure, and it uses ground reinforcement techniques that would not be unstable during any disaster that causes landslides. Such soil improvement techniques are more cost-effective and easier to construct than other options, such as structural strengthening, are. For wooden bridges, resistance to landslide movement can be improved by several measures, as outlined in the following bullet list. Soil improvement and strengthening, as proposed in the first bullet, increase the stability and the capacity of soil that surrounds bridges (FHWA 2006 & ODOT 2012). This method improves weak soils by increasing their resisting capacity and by reducing the driving force.

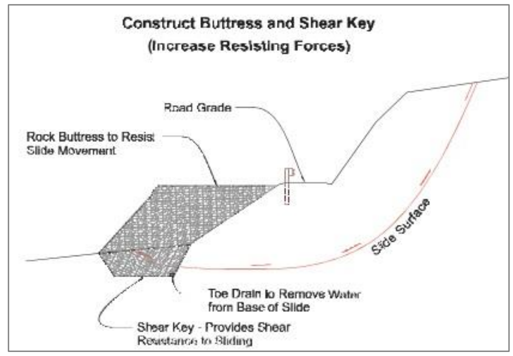
It is assumed that the standard-QA cost is about 10% of the project budget, and the higher-level QA cost is approximately 15% of the total budget for wooden bridge construction. Therefore, for better QA of retrofit work, the higher-level QA approach is considered in this report, as listed in the last bullet, and the standard QA is assumed to have been applied to the existing components.

Improvements to increase the landslide resistance of wooden bridges include:

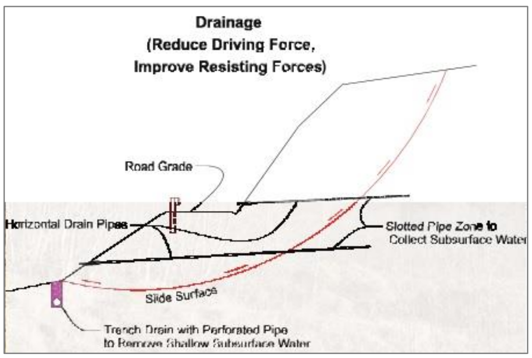
- Apply soil improvement and strengthening by several measures (e.g., retaining walls, buttresses and shear keys, drainage, and a stable slope angle); see Figure 153.
- Strengthen the bridge piers and foundation to resist and to accommodate the ground movements.
- Assess geotechnical components and strengthen them if they have any landslide deficiencies.
- Perform routine and regular maintenance for all components and fix any observed problems.
- Conduct the higher-level QA protocol (e.g., daily testing and continual and detailed field inspection).



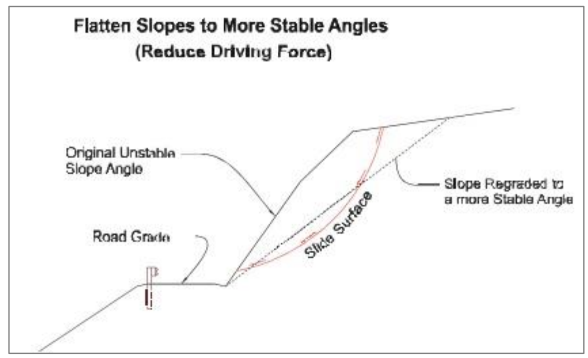

Retaining wall




Buttress and shear key




Drainage




Stable slope angle

Figure 153. Examples of soil improvement against landslide hazard for wooden bridges

### 5.6.8.4 Cost-benefit considerations

A fragility function for PGD was developed for roadway bridges that are built in the United States (FEMA 2013a), and the fragility curve can be used to express the probability of exceeding complete damage and the horizontal ground displacement. For wooden bridges, the fragility curve is modified to adjust for older bridge age and soil deterioration over a longer period, as Figure 154 shows. By using this fragility curve, based on the improvement methods that are proposed in Section 5.6.8.3, it is expected that the possible landslide displacement will be greatly reduced. Therefore, it is assumed that the ground displacement due to a certain level of landslide is 14 in. before any soil improvements, and that ground displacement after soil improvements is reduced to 7 in. (i.e., reduced by half). The fragility parameters of the function are then applied to estimate the damage probability (i.e., resiliency index) for a wooden bridge from landslide. For displacement with a PGD of 14 in. before any improvements, the probability of exceeding the complete damage state (DS4) is 63%. The PGD of 7 in. after implementation of soil improvements results in a 25% probability of complete damage.

The improvement cost greatly depends on the characteristics, the area, the volume of the soil and the local ground situation, but in this report, the cost to improve the landslide hazard resistance (i.e., soil improvements) is assumed to be an average of 25% of the replacement cost of wooden bridge itself. The total improvement cost, including the higher-level QA, is then estimated as 25% of the total cost of wooden bridge construction. Thus, wooden bridge damage due to a landslide is expected to be minor after soil improvements, so the restoration of a wooden bridge will be completed in less time. Such expedited recovery helps avoid the considerable loss of economic and public benefits.

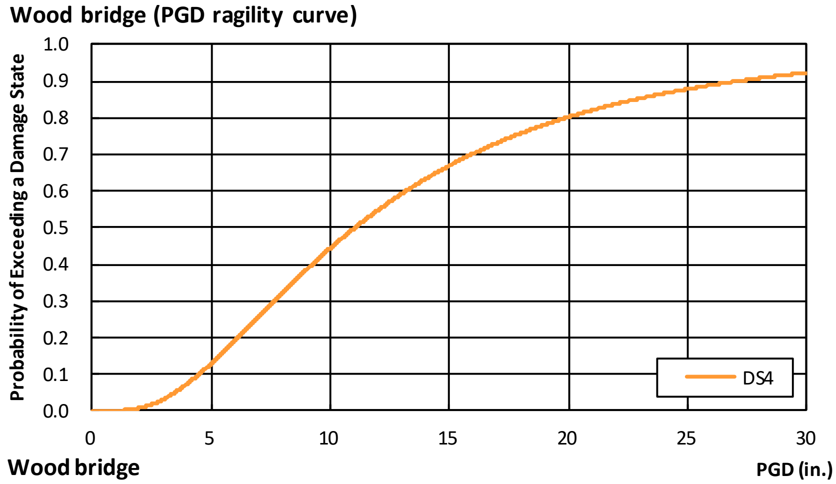



Figure 154. Horizontal displacement fragility functions for wooden bridges

# 6. REFERENCES

## 6.1 Power infrastructure


Ahmed, K., Sayed, M. and Kim, D. (2015) *Seismic Fragility of Base-isolated Nuclear Power Plant Structures Considering Spatially Varying Ground Motions,* Proceedings of SMIRT 23, Manchester, UK

Basu, P., Ravindra, M., and Mihara, Y. (2015) *Component fragility for use in PSA of Nuclear Power Plants,* Proceedings of SMIRT 23, Manchester, UK

Bensi, M., Ferrante, F., and Philip, J. (2015) *Assessment of Flood Fragility for Nuclear Power Plants: Challenges and Next Steps*, Proceedings of SMIRT 23, Manchester, UK

Coleman, H., Wei, C., and Lindell J. (2004) *Hydraulic Design of Spillways*

Doohyun, Kyung, D, Choi, Y., Jeong S., and Lee, J (2015) *Improved Performance of Electrical Transmission Tower Structure Using Connected Foundation in Soft Ground*, Energies 2015, 8, 4963-4982

Eidinger J., Pratt, D., and Nikman, F. (2008) *Seismic and Wind Design for Solar Panels,* Proceedings of the 14th World Conference on Earthquake Engineering, Beijing, China.

Eidinger, J., & Davis, C. A. (2012). *Recent Earthquakes: Implications for U.S. Water Utilities*. Water Research Foundation.

EPRI (2009) *Seismic Fragility Application Guide Update, 1019200*, Electric Power Research Institute, CA, USA.

FEMA (2013a) Hazus-MH2.1, *Multihazard Loss Estimation Methodology*, *Earthquake Model Technical Manual*, Federal Emergency Management Agency, Washington DC, USA.

FEMA (2013b) Hazus-MH2.1, *Multihazard Loss Estimation Methodology*, *Hurricane Model Technical Manual*, Federal Emergency Management Agency, Washington DC, USA.

FEMA (2013c) Hazus-MH2.1, *Multihazard Loss Estimation Methodology, Flood Model Technical Manual,* Federal Emergency Management Agency, Washington DC, USA.

Ghanaat, Y., Patev, R., and Chudgar, A. (2012) *Seismic fragility analysis of concrete dams*, Proceedings of the 15th World Conference on Earthquake Engineering, Lisbon, Portugal.

Ham, H., Lee, S., and Kim, H. (2009) *Development of Typhoon Fragility for Industrial Buildings*, The Seventh Asia-Pacific Conference on Wind Engineering, Taipei, Taiwan, China

Lopez, A., Rocha, L, Escobedo, D, and Semsa J (2009) *Reliability and Vulnerability Analysis of Electrical Substations and Transmission Towers for Definition of Wind and Seismic Damage Maps for Mexico*, 11th America Conference on Wind Engineering, San Juan, Puerto Rico.

Myers, A., Gupta A., and Ramirez, C. (2012) *Evaluation of the Seismic Vulnerability of Tubular Wind Turbine Towers*, Proceedings of the 15th World Conference on Earthquake Engineering, Lisbon, Portugal.

Sutherland, H. (1999) *On the Fatigue Analysis of Wind Turbines*, Report No. SAND99-0089, Sandia National Laboratories, Albuquerque, NM, USA.

Transpower (2011). *22 February 2011 Christchurch Earthquake. Key Findings and Lessons Learned*, Wellington, NZ.

University of Naples Federico. (2012) *Seismic fragility analysis of concrete dams*, Proceedings of the 15th World Conference on Earthquake Engineering, Lisbon, Portugal.

Vorogushyn, S., Merz, B. and Apel, H. (2009) *Development of dike fragility curves for piping and micro-instability breach mechanisms*, Natural Hazards and Earth System Sciences.

Wironen, A., Butler, T., Massicotte, P. (2013) *Substation foundations designed to stand up to earthquake soil loading*, Transmission and Distribution World.

Yu, C., Bolisetti, C., Coleman, J., Kosbab, B. and Whittaker, A. (2018) *Using seismic isolation to reduce risk and capital cost of safety-related nuclear structures*, Nuclear Engineering and Design.

## 6.2 Transport infrastructure


American Railway Engineering and Maintenance-of-Way Association (AREMA) (2018) "Manual for Railway Engineering", American Railway Engineering and Maintenance-of-Way Association, Lanham, MD, USA.

California Department of Transportation (Caltrans) (2008) "Socio-Economic Effect of Seismic Retrofit Implemented on Bridges in the Los Angeles Highway Network", California Department of Transportation Engineering Services Center, Sacramento, CA, USA.

Day, K.R., and Barkan, C.P.L. (2002) "The Potential Impact of a Mid-America Earthquake on the Railroad Network," Proceedings of the American Railway Engineering and Maintenance-of-Way Association, Washington, DC, USA.

"Estimating Wind Speeds With Visual Clues" National Weather Service Portland, USA. ftp://aidea.org/REFund/Round%206/Applications/966_High-penetration%20Wind%20Energy%20Project-%20Kokhanok/Kokhanok%20OEM/Volume_1/9.0%20Turbine%20Information/9.007%20Wind%20Speed%20Indicator%20Chart.pdf

Federal Emergency Management Agency (FEMA) (1999) "FEMA 348, Protecting Building Utilities from Flood Damage", Federal Emergency Management Agency, Washington, DC, USA.

Federal Emergency Management Agency (FEMA) (2007) "FEMA 543, Design Guide for Improving Critical Facility Safety from Flooding and High Winds", Federal Emergency Management Agency, Washington, DC, USA.

Federal Emergency Management Agency (FEMA) (2010) "FEMA P-804, Wind Retrofit Guide for Residential Buildings", Federal Emergency Management Agency, Washington, DC, USA.

Federal Emergency Management Agency (FEMA) (2013d) "Reducing Flood Effects in Critical Facilities: HURRICANE SANDY RECOVERY ADVISORY", Federal Emergency Management Agency, Washington, DC, USA.

Ferritto, J.M. (1997) "Seismic Design Criteria for Soil Liquefaction", Naval Facilities Engineering Service Center, Port Hueneme, CA, USA.

Huizinga, J., Moel, H. & Szewczyk, W. (2014) "Support for Creating Flood Damage Business Continuity Plan (BCP)", European Union, Luxembourg.

Iowa Department of Transportation (DOT) (2014) "Upper Midwest Transportation Hub Manly, Iowa", Iowa Department of Transportation, Ames, IA, USA.

Japan Meteorological Agency (JMA seismic), "Commentary table explaining the JMA Seismic Intensity Scale", Japan Meteorological Agency, Tokyo, Japan. https://www.jma.go.jp/jma/kishou/know/shindo/kaisetsu.html.

Japan Meteorological Agency (JMA wind), "Intensity, Speed, Effects and Gust Speed of Wind", Japan Meteorological Agency, Tokyo, Japan . https://www.jma.go.jp/jma/kishou/know/yougo_hp/kazehyo.pdf.

Koshimura, S., Namegaya, Y. & Yanagisawa, H. (2009) "Tsunami Fragility, A New Measure to Identify Tsunami Damage, Journal of Disaster Research, Vol. 4, No. 6, 2009, pp. 479-488.

Nakashima, Y., Matsumoto, H., Sugai, T., Yamazaki, H., Nakamura, T. & Mochizuki, T. (2015) "A Study on Adoption of the Reliability-based Design in the Earthquake Resistance of the Airport", Technical Note of National Institute for Land and Infrastructure Management, Tokyo, Japan.

National Severe Storm Laboratory (NSSL), “Severe Weather 101: Damaging Winds Basics”, National Oceanic and Atmospheric Administration (NOAA), Norman, OK, USA. https://www.nssl.noaa.gov/education/svrwx101/wind/.

New York City (NYC) (2013) “A Stronger, More Resilient New York”, New York City, New York City, NY, 2013.

Takeo Office of River (MLIT) (2014) “Support for Creating Flood Damage Business Continuity Plan (BCP)”, Ministry of Land, Infrastructure, Transportation and Tourism, Kyushu Regional Development Bureau, Fukuoka, Japan http://www.qsr.mlit.go.jp/takeo/tiikibousai/kigyou/bcp.html.

The New York Times (NYT) (2012) “Assessing the Damage From Hurricane Sandy”, The New York Times, New York City, NY, October 29th, 2012.

U.S. Army Corps of Engineers (2017) “Planning Risk Assessment for Flood Risk Management Studies”, Department of the Army, Washington, DC, USA.

United States Geological Survey (USGS seismic), “The Modified Mercalli Intensity Scale”, United States Geological Survey, Reston, VA, USA. https://earthquake.usgs.gov/learn/topics/mercalli.php.

Urayasu City (2012) “Performance Objectives for Soil Liquefaction”, Government of Urayasu City, Urayasu City, Chiba, Japan. https://www.city.urayasu.lg.jp/_res/projects/default_project/_page_/001/005/444/betten1-5-1mokuhyo.pdf.

U.S. Department of Transportation Federal Highway Administration (FHWA) (2016) “Analysis of Construction Quality Assurance Procedures on Federally Funded Local Public Agency Projects”, US Department of Transportation, McLean, VA, USA.

### 6.3 Water infrastructure


ALA (2001b) *Seismic Fragility Formulations for Water Systems Part 2 – Appendices*, American Lifelines Alliance, American Society of Civil Engineers, VA, USA.

American Lifelines Alliance (ALA) (2001a) *Seismic Fragility Formulations for Water Systems Part 1 – Guideline*, American Lifelines Alliance, American Society of Civil Engineers, VA, USA.

American Society of Civil Engineers (ASCE) (2015) *Flood Resistant Design and Construction* (ASCE/SEI 24-14).

Andrus, R.D. & Chung, R.M. (1995) "Ground Improvement Techniques for Liquefaction Remediation Near Existing Lifelines", Building and Fire Research Laboratory, National Institute of Standards and Technology, Gaithersburg, MD, USA.

California Geological Survey (CGS) (2008) *Guidelines for Evaluating and Mitigating Seismic Hazards in California*, California Department of Conservation, Sacramento, CA, USA.

Davis, C.A., and Castruita, J. (2013). Los Angeles Pilot Project Using Japanese Earthquake Resistant Joint Ductile Iron Pipe. Presentation at California-Nevada Section, American Water Works Association, October 1, 2013.

"Estimating Wind Speeds with Visual Clues". National Weather Service Portland, USA. ftp://aidea.org/REFund/Round%206/Applications/966_High-penetration%20Wind%20Energy%20Project-%20Kokhanok/Kokhanok%20OEM/Volume_1/9.0%20Turbine%20Information/9.007%20Wind%20Speed%20Indicator%20Chart.pdf.

Federal Emergency Management Agency (FEMA) (1999) "FEMA 348, Protecting Building Utilities from Flood Damage", Federal Emergency Management Agency, Washington, DC, USA.

Federal Emergency Management Agency (FEMA) (2002) "FEMA 412, Installing Seismic Restraints for Mechanical Equipment", Federal Emergency Management Agency, Washington, DC, USA.

Federal Emergency Management Agency (FEMA) (2004) "FEMA 413, Installing Seismic Restraints for Electrical Equipment", Federal Emergency Management Agency, Washington, DC, USA.

Federal Emergency Management Agency (FEMA) (2006) "FEMA 547, Techniques for the Seismic Rehabilitation of Existing Buildings", Federal Emergency Management Agency, Washington, DC, USA.

Federal Emergency Management Agency (FEMA) (2007) "FEMA 543, Design Guide for Improving Critical Facility Safety from Flooding and High Winds", Federal Emergency Management Agency, Washington, DC, USA.

Federal Emergency Management Agency (FEMA) (2013d) "Reducing Flood Effects in Critical Facilities: HURRICANE SANDY RECOVERY ADVISORY", Federal Emergency Management Agency, Washington, DC, USA.

FEMA (2005) Federal Emergency Management Agency (FEMA) and National Institute of Building Science (NIBS), 2005, "American Lifelines Alliance: Seismic Guideline for Water Pipelines March 2005", Federal Emergency Management Agency, Washington, DC, USA.

FEMA (2008) *2006 Evaluation of the National Flood Insurance Program's Building Standards*, Federal Emergency Management Agency, Washington DC, USA.

FEMA (2013a) Hazus-MH2.1, *Multihazard Loss Estimation Methodology, Earthquake Model Technical Manual*, Federal Emergency Management Agency, Washington DC, USA.

FEMA (2013b) Hazus-MH2.1, *Multihazard Loss Estimation Methodology, Hurricane Model Technical Manual*, Federal Emergency Management Agency, Washington DC, USA.

FEMA (2013c) Hazus-MH2.1, *Multihazard Loss Estimation Methodology, Flood Model Technical Manual*, Federal Emergency Management Agency, Washington DC, USA.

FEMA 543. (2007) Design Guide for Improving Critical Facility Safety from Flooding and High Winds.

FEMA (2013) Hurricane Sandy Recovery Advisories and Fact Sheets: RA2. Reducing Flood Effects in Critical Facilities.

Ferritto, J.M. (1997) “Seismic Design Criteria for Soil Liquefaction”, Naval Facilities Engineering Service Center, Port Hueneme, CA, USA.

Haddaway (2015) Earthquake-Resistant Ductile Iron Pipe Makes U.S. Debut in Los Angeles, Waterworld, https://www.waterworld.com/articles/print/volume-31/issue-4/features/earthquake-resistant-ductile-iron-pipe-makes-u-s-debut-in-los-angeles.html

Huizinga, J., Moel, H. & Szewczyk, W. (2014) “Support for Creating Flood Damage Business Continuity Plan (BCP)”, European Union, Luxembourg.

Matsuhashi, M., Tsushima, I., Fukatani, W., and Yokota, T. (2014). Damage to sewage systems caused by the Great East Japan Earthquake, and governmental policy, Soils and Foundations, Volume 54, Issue 4, 2014, Pages 902-909, ISSN 0038-0806, https://doi.org/10.1016/j.sandf.2014.06.019. (http://www.sciencedirect.com/science/article/pii/S003808061400081X)

Mejia, L. (2005) *Performance-Based Seismic Retrofit of Embankment Dams to Mitigate Liquefaction*, ASCE Geo-Frontiers 2005 Specialty Conference, Austin, Texas, USA.

Nakashima, Y., Matsumoto, H., Sugai, T., Yamazaki, H., Nakamura, T. & Mochizuki, T. (2015) “A Study on Adoption of the Reliability-based Design in the Earthquake Resistance of the Airport”, Technical Note of National Institute for Land and Infrastructure Management, Tokyo, Japan.

National Severe Storm Laboratory (NSSL), “Severe Weather 101: Damaging Winds Basics”, National Oceanic and Atmospheric Administration (NOAA), Norman, OK, USA. https://www.nssl.noaa.gov/education/svrwx101/wind/.

NFPA (2018) NFPA 22 *Standard for Water Tanks for Private Fire Protection*, national Fire Protection Agency, Qunicy, MA, USA.

Panico, A., Iasimoneb F., Fabbrocinob, G., and Pirozzic, F. (2015) *Vulnerability Assessment of Drinking Water Treatment Plants*, Chemical Engineering Transactions, The Italian Association of Chemical Engineering.

Pells, S. and Fell, R. (2003) *Damage and cracking of embankment dams by earthquake and the implications for internal erosion and piping* Proc., 21st Internal Congress on Large Dams, Montreal. ICOLD, Paris Q83-R17, International Commission on Large Dams, Paris, France.

Saez, E. and Ledezma, C., (2014) *Liquefaction mitigation using secant piles wall under a large water tank,* Volume 79, Part B, , Pages 415-428, Soil Dynamics and Earthquake Engineering.

Swaisgood, J. R. (2014). *Behavior of embankment dams during earthquake,* Journal of Dam Safety, ASDSO, 12(2): pp 35-44.

Takeo Office of River (MLIT) (2014) “Support for Creating Flood Damage Business Continuity Plan (BCP)”, Ministry of Land, Infrastructure, Transportation and Tourism, Kyushu Regional Development Bureau, Fukuoka, Japan. http://www.qsr.mlit.go.jp/takeo/tiikibousai/kigyou/bcp.html.

Tansel B. and Ahmed N., (2000) *Structural Stability of Elevated Water Reservoirs Under Hurricane Force Wind Conditions*, Proceedings of the 8th ASCE Specialty Conference on Probabilistic Mechanics and Structural Reliability.

U.S. Army Corps of Engineers (2017) "Planning Risk Assessment for Flood Risk Management Studies", Department of the Army, Washington, DC, USA.

U.S. Department of the Interior, USDI (2012), *Design Standards No. 13, Embankment Dams, Chapter 6: Freeboard*, U.S. Department of the Interior, Bureau of Reclamation, Washington DC, USA.

United States Environmental Protection Agency (EPA) (2018) "Earthquake Resilience Guide for Water and Wastewater Utilities", EPA Office of Water (4608T), USA.

United States Geological Survey (USGS seismic), "The Modified Mercalli Intensity Scale", United States Geological Survey, Reston, VA, USA. https://earthquake.usgs.gov/learn/topics/mercalli.php.

Urayasu City (2012) "Performance Objectives for Soil Liquefaction", Government of Urayasu City, Urayasu City, Chiba, Japan. https://www.city.urayasu.lg.jp/_res/projects/default_project/_page_/001/005/444/betten1-5-1mokuhyo.pdf.

Yoshizawa, M., Onimaru, S., Uchida, A., and Hatanaka, M. (2000) *A Study on Liquefied Ground Disruption Effects on Liquid Storage Tank* Behavior, 12th World Conference on Earthquake Engineering, Vancouver, Canada.

## 6.4 Roadway infrastructure


AASHTO (2011) *AASHTO Guide Specifications for LRFD Seismic Bridge Design*, American Association of State Highway and Transportation Officials, Washington DC, USA.

AASHTO (2017) *AASHTO LRFD Bridge Design Specifications*, American Association of State Highway and Transportation Officials, Washington DC, USA.

Alemdar, F., Nagati, D. Matamoros, A., Bennett, C., and Rolfe, S. (2014)*Repairing Distortion-Induced Fatigue Cracks in Steel Bridge Girders Using Angles-with-Plate Retrofit Technique. I: Physical Simulations*, Journal of Structural Engineering, American Society of Civil Engineers, VA, USA

Applied Technology Council, "Earthquake Damage Evaluation Data for California", ATC-13, Redwood City, CA, 1985.

Barry (2014) *Cost Savings by Using Geosynthetics in the Construction of Civil Works Projects*, Deutsche Gesellschaft für Geotechnik (German Geotechnical Society) (DGGT), Berlin, Germany.

Caltrans (2018) Caltrans Climate Change Vulnerability Assessments: District 4 Technical Report. January 2018.

FEMA (2006) *Seismic Retrofitting Manual for Highway Structures: Part 2 – Retaining Structures, Slopes, Tunnels, Culverts, and Roadways*, Federal Highway Administration, McLean VA, USA.

FEMA (2013a) Hazus-MH2.1, *Multihazard Loss Estimation Methodology, Earthquake Model Technical Manual*, Federal Emergency Management Agency, Washington DC, USA.

FEMA (2013b) Hazus-MH2.1, *Multihazard Loss Estimation Methodology, Hurricane Model Technical Manual*, Federal Emergency Management Agency, Washington DC, USA.

FEMA (2013c) Hazus-MH2.1, *Multihazard Loss Estimation Methodology, Flood Model Technical Manual*, Federal Emergency Management Agency, Washington DC, USA.

FHWA (2009) *Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance-Third Edition*, Hydraulic Engineering Circular (HEC) No. 23, Federal Highway Administration, Washington DC, USA.

Mason, D., Brabhaharan, P. & Saul, G. (2017) Performance of road networks in the 2016 Kaikōura earthquake: Observations on ground damage and outage effects. Proc. 20th NZGS Geotechnical Symposium. Eds. GJ Alexander & CY Chin, Napier.

Ogorzalek, K.A., Dilsworth, B.W., and Takhirov, S.M. (2017) "Experimental Program for Historic and Contemporary Strengthening of Wood Connections and Advanced Measurements by Laser Scanning and Image Tracking", 3rd International Conference on Protection of Historical Constructions, Lisbon, Portugal, 12-15 July 2017.

Oregon Department of Transportation (ODOT) (2012) "Oregon Highways Seismic Options Report", Bridge and Geo-Environmental Sections, Technical Services Branch, Oregon Department of Transportation, Salem, OR, USA.

US Department of Transportation Federal Highway Administration (FHWA) (2006) "Seismic Retrofitting Manual for Highway Structures: Part 1 - Bridges", US Department of Transportation, McLean, VA, USA.

US Department of Transportation Federal Highway Administration (FHWA) (2005) "Covered Bridge Manual", US Department of Transportation, McLean, VA, USA.

## Appendix A - Assessment of landslide damage for railways

### A.1 Overview

Although not part of the scope of the project, a brief discussion regarding the landslide hazard for the railways and mitigation measures are presented in this section. It is noted that the appendix is not intended to provide a detailed description of the subject matter, but rather provide some general information.

### A.2 Summary

The results from a literature review of highways are summarized in Table A.1. The following subsections of this background report provide more detailed descriptions and background information.

| Hazard | Susceptible | Improvements |
|---|---|---|
| **Landslide** | Y | Rock shed |

Table A.1. Summary of findings for railways subject to landslide

### A.3 Vulnerability to natural hazards

Railways in mountainous regions have experienced damage in past landslides. Figure A.1 shows an example. The landslide is typically caused by either an earthquake or a heavy rain leading to saturation of soil (and thus soil instability).


Landslide (Switzerland, 2014)

Figure A.1. Damage to railways from landslide

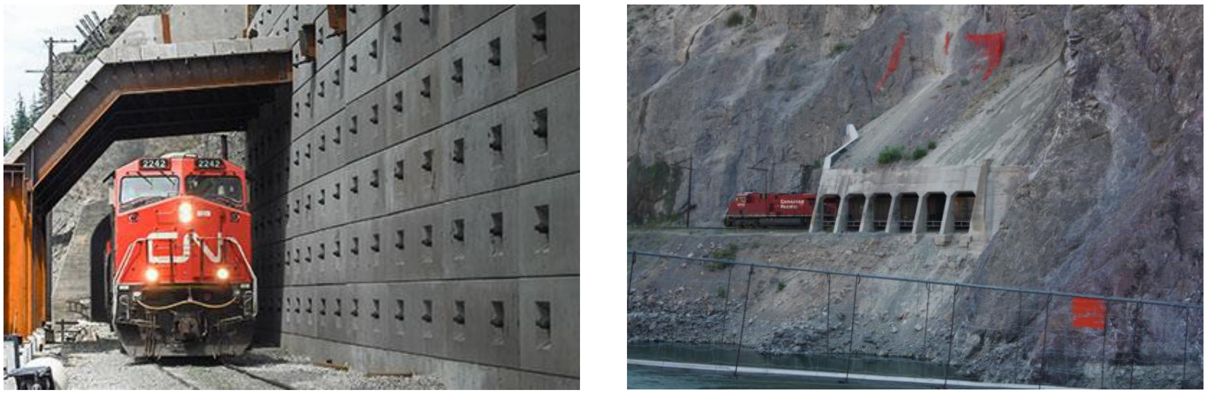
### A.4 Key metric for consideration

Railways in mountainous regions are susceptible to loss of life, economic costs as a result of damage, and service interruption that can last months because of landslides. The key metric to consider is the relative cost of performing no capital improvement with the initial cost of these improvements. The cost of no improvement includes continuous monitoring, maintenance, cost of less robust mitigation, and the economic cost of loss of operation and track repair.

### A.5 Infrastructure improvements

Soil improvement, an improvement method used in landslide mitigation, might not always be feasible because of lack of access and the terrain. For new construction, tunnels or track re-routing could be considered. For both existing and new tracks, an option would be to provide a bypass for the displaced soil which allows for the safe operation of the rail line even for large ground displacement. An example is as follows:

- Use rock shed (see Figure A.2), which is essentially maintenance free.



Steel-frameRockshed (Canada) Concrete-frame rockshed (Canada)

Figure A.2. Examples of landslide mitigation

### A.6 Cost-benefit considerations

Compass International (2017) provides the benchmark cost for new rail construction. Pertinent data adopted from this source is summarized in Table A.2. Including other capital improvements (culverts, etc.), the cost per km is approximately US $1,500,000.

| Item | Average cost $US/km |
|---|---|
| Single track on stone bed | 1,100,000 |
| Central control system | 170,000 |

Table A.2. New railroad construction cost

Caltrans (2014) completed the construction of the Pitkins Rock shed. The cost benefit analysis adopted from the report is summarized in Table A.3.

| Item | Do not improve | Build rock shed (US $) |
|---|---|---|
| Construction cost | -- | $30,000,000 |
| Maintenance cost | $112,000,000 | $2,000,000 |
| Total cost | $112,000,000 | $32,000,000 |

Table A.3. Cost benefit analysis for an example rock shed

### A.7 References

Caltrans (2014) *Why we need a bridge at Pitkins Curve and a rock shed at Rain Rocks.* State of California Department of Transportation. USA.

Compass International (2017) "2017 Railroad Engineering & Construction Cost Benchmarks" https://compassinternational.net/railroad-engineering-construction-cost-benchmarks/